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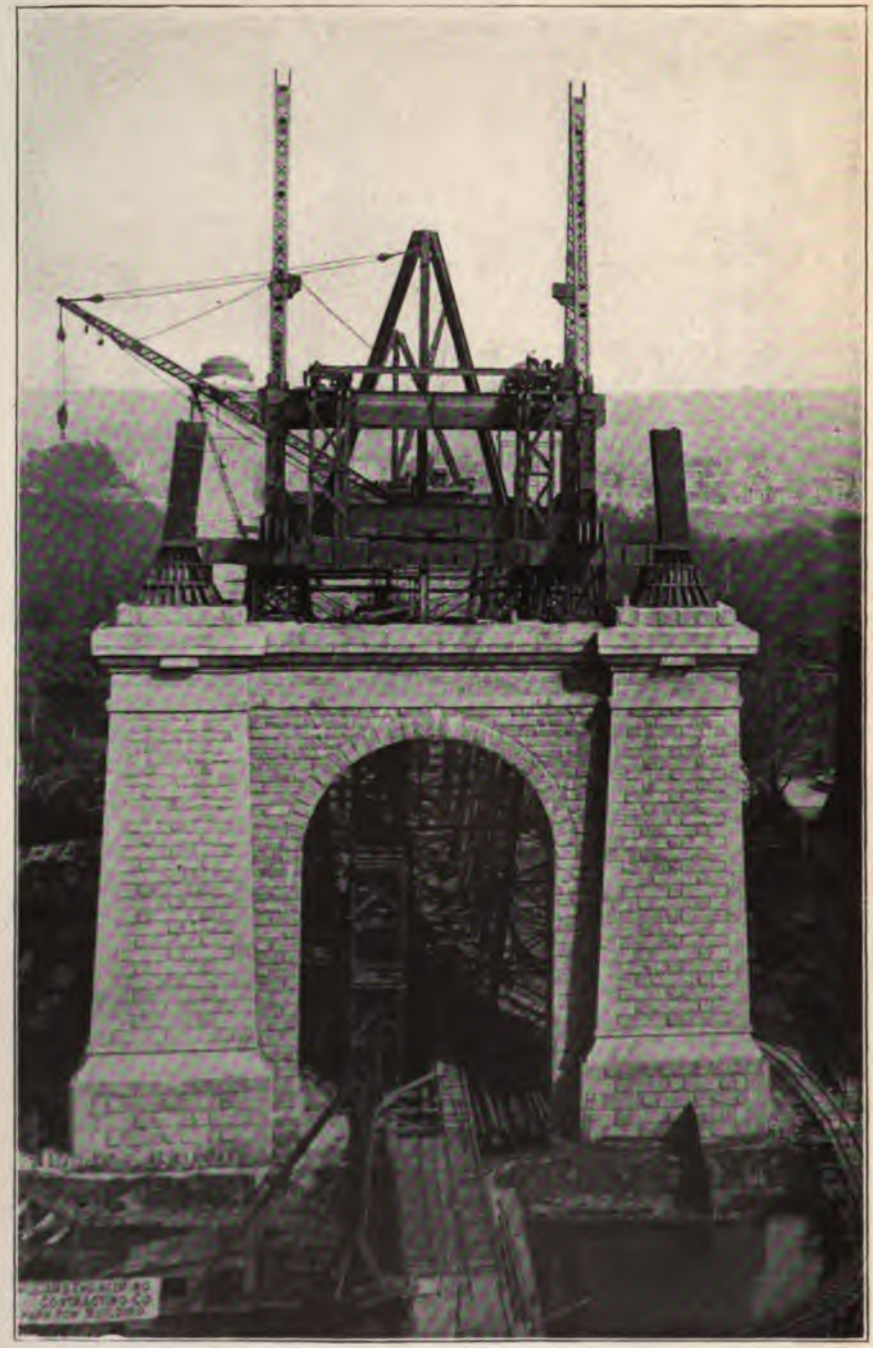

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of

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A General Reference Work

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CONSTRUCTION, HYDRAULIC ENGINEERING, IRRIGATION,
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THE editors have freely consulted the standard technical literature of America and Europe in the preparation of these volumes. They desire to express their indebtedness, particularly, to the following eminent authorities, whose well-known treatises should be in the library of everyone interested in Civil Engineering.

Grateful acknowledgment is here made also for the invaluable co-operation of the foremost Civil, Structural, Railroad, Hydraulic, and Sanitary Engineers in making these volumes thoroughly representative of the very best and latest practice in every branch of the broad field of Civil Engineering; also for the valuable drawings and data, illustrations, suggestions, criticisms, and other courtesies.

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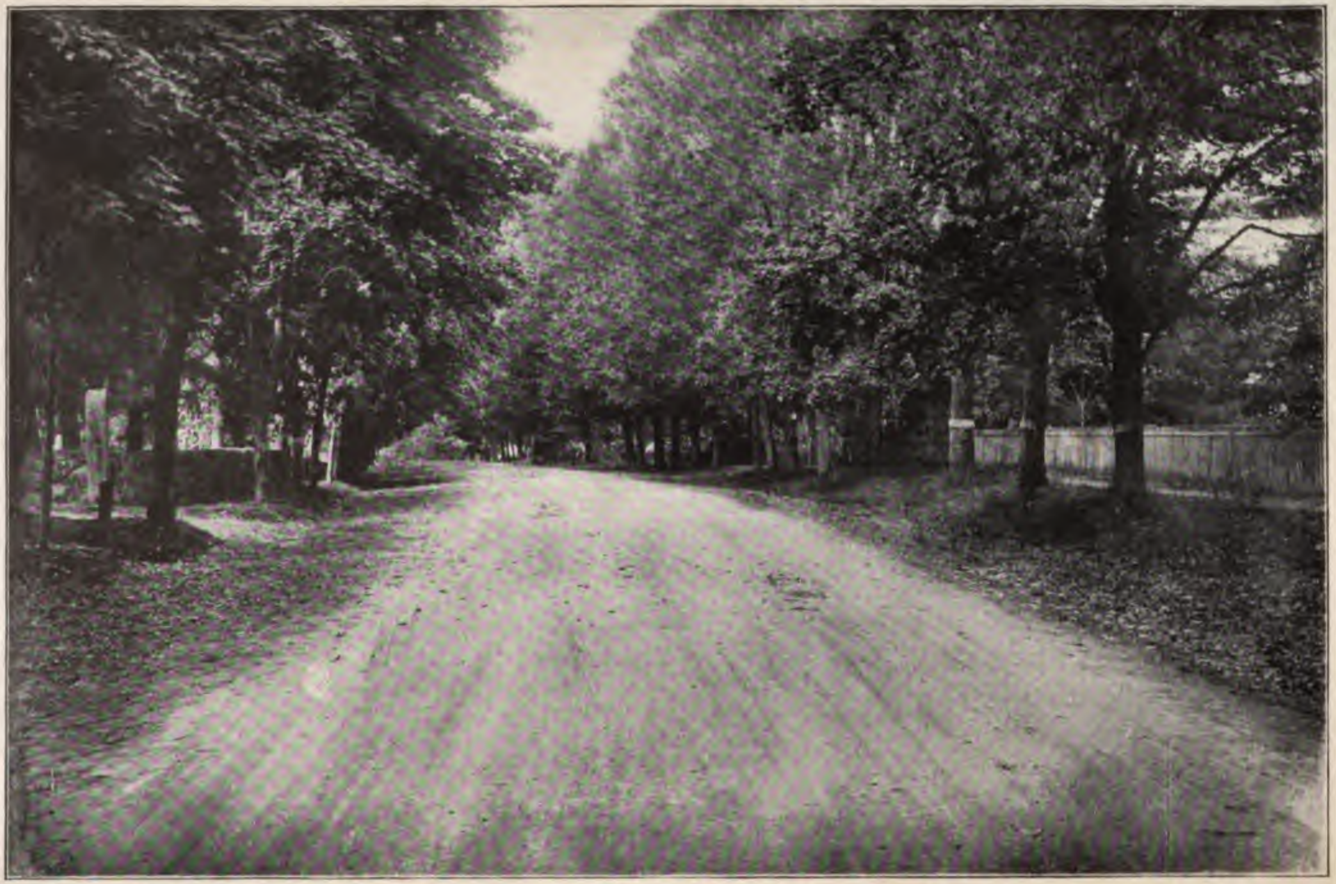
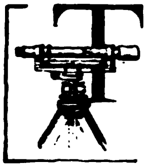


STATE HIGHWAY AT WESTON, MASSACHUSETTS

Showing road after tar treatment and after several weeks' use. Such a roadway is practically free from dust, and motor vehicles do not erode its surface so rapidly.

Courtesy of Massachusetts Highway Commission.

Foreword



THE marvelous developments of the present day in the field of Civil Engineering, as seen in the extension of railroad lines, the improvement of highways and waterways, the increasing application of steel and reinforced concrete to construction work, the development of water power and irrigation projects, etc., have created a distinct necessity for an authoritative work of general reference embodying the results and methods of the latest engineering achievement. The Cyclopedia of Civil Engineering is designed to fill this acknowledged need.

¶ The aim of the publishers has been to create a work which, while adequate to meet all demands of the technically trained expert, will appeal equally to the self-taught practical man, who, as a result of the unavoidable conditions of his environment, may be denied the advantages of training at a resident technical school. The Cyclopedia covers not only the fundamentals that underlie all civil engineering, but their application to all types of engineering problems; and, by placing the reader in direct contact with the experience of teachers fresh from practical work, furnishes him that adjustment to advanced modern needs and conditions which is a necessity even to the technical graduate.

¶ The Cyclopedia of Civil Engineering is a compilation of representative Instruction Books of the American School of Correspondence, and is based upon the method which this school has developed and effectively used for many years in teaching the principles and practice of engineering in its different branches. The success attained by this institution as a factor in the machinery of modern technical education is in itself the best possible guarantee for the present work.

¶ Therefore, while these volumes are a marked innovation in technical literature — representing, as they do, the best ideas and methods of a large number of *different* authors, each an acknowledged authority in his work — they are by no means an experiment, but are in fact based on what long experience has demonstrated to be the best method yet devised for the education of the busy workingman. They have been prepared only after the most careful study of modern needs as developed under conditions of actual practice at engineering headquarters and in the field.

¶ Grateful acknowledgment is due the corps of authors and collaborators — engineers of wide practical experience, and teachers of well-recognized ability — without whose co-operation this work would have been impossible.

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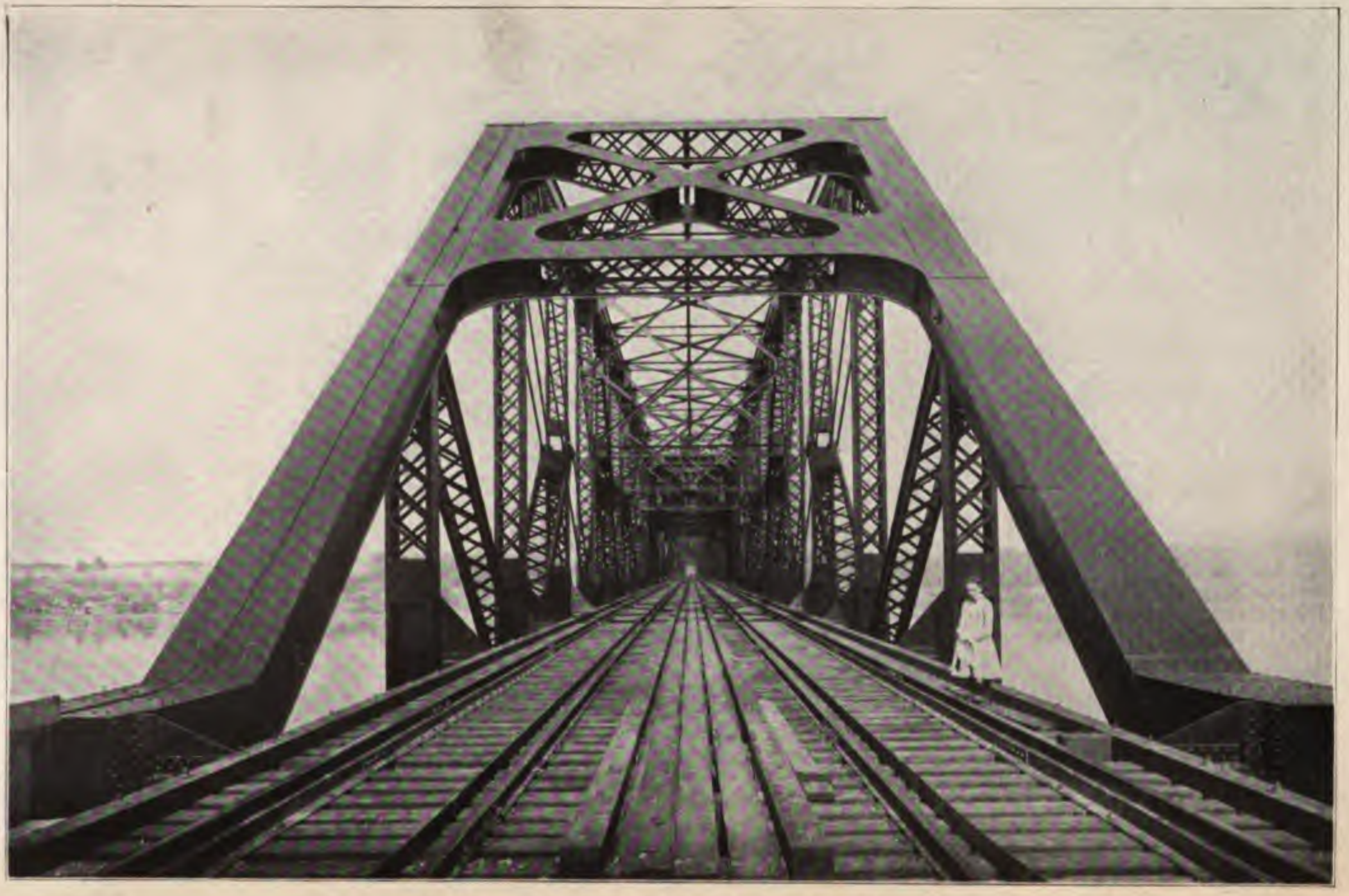

BRIDGE OVER MISSISSIPPI RIVER AT THEBES, ILLINOIS

Noble & Modjeski, Engineers, Chicago, Ill. View looking through from Missouri end.

BRIDGE ENGINEERING

PART I

BRIDGE ANALYSIS

1. Introduction. The following treatment of the subject of Bridge Analysis, while not exhaustive, is regarded as sufficiently elaborate to develop and instill the principal theoretical considerations, to illustrate the most convenient and practical methods of analyzing the common forms of trusses and girders, and also to lay a sufficient foundation for the analysis of such other trusses as are not specifically mentioned or treated herein.

The necessary steps and operations required for a proper analysis of the several types of bridges are fully demonstrated by sketches and computations, the numerical values being mechanically obtained by the use of a *slide rule*, which is a handy instrument for quickly performing the operations of multiplication and division, and for squaring and extracting the roots of numbers. The values given may differ from the exact value by one unit in the second decimal place (seldom more) and are sufficiently accurate for the purpose of design. All bridge computers should be proficient in the use of the slide rule.

The problems given in the back of this Instruction Paper, exemplifying the practical application of the subject-matter treated in the various articles, should be solved by the student as each article is mastered.

HISTORY

2. Early Bridges. Early bridges were not bridges according to the present conception of the term. They were simple pile trestle bents placed at frequent intervals and connected by wooden beams on which the floor was placed. The *Pons sublicius*, built over the Tiber, at Rome, about 650 years before Christ was born, was of this trestle type. Also the famous bridge built by Cæsar across the Rhine in 55 B. C. was of the same kind of construction. As civilization progressed, the arch type was developed; and in 1390 the great

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bridge at Trezzo over the River Adda was built of one span of 251 feet, which has never been eclipsed or equaled.

3. **Truss Bridge Development.** The first truss bridge is supposed to have been originated by Palladio, an Italian, who used the king-post truss (Fig. 1) about 1570. Its importance was not recognized, and it became entirely forgotten until it was rediscovered in 1798 by Theodore Burr, an American, who used it in his construction. About the same time, Burr invented the truss that bears his name, which was in reality a series of king-post trusses (see Fig. 8). This was found to be unstable under moving loads, and was therefore stiffened by the use of an arch (Fig. 2), or was built somewhat as an arch, there being considerable rise at the center of the span (Fig. 3). By 1830 the principle of the double cross-bracing in the panel was understood; and in quick succession came the patents of Long, Howe, Pratt, and Whipple on forms of trusses which bear their respective names.

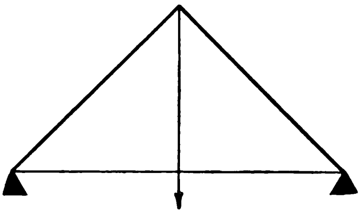


Fig. 1. King-Post Truss.

It remained for Squire Whipple in 1847 to place the science of bridge building on a rational and exact mathematical basis such as is now used. Previous to this time, and indeed several years afterwards—for Whipple's work did not become generally known until a much later date—bridges were built, not from previously computed strains, but by "judgment." All parts of a bridge were made of the same size, and if one started to fail it was replaced by a larger one; or small models were made and loaded proportionally, broken members being replaced by larger ones. There is no doubt

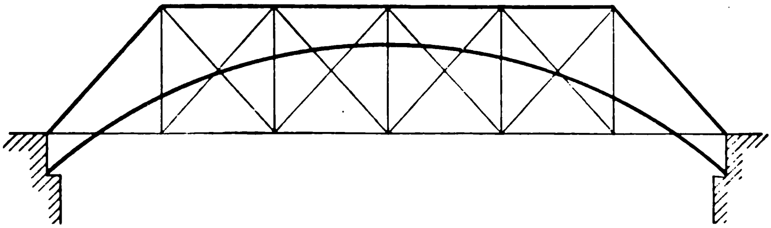


Fig. 2. King-Post Truss Bridge Stiffened by Arch.

that many of the bridges built at this period were very weak as well as very strong. The failures are not remembered; but the sound judgment of many of our earlier bridge engineers is recorded in the wooden structures they left behind them, some of which have stood the demands of traffic for over a century. After 1850, bridges were built from computed stresses; wood was discarded; and the development became rapid, until about 1870, when the introduction of subdiagonal systems brought the truss system to practically what it is to-day.

DEFINITIONS AND DESCRIPTIONS

4. **Trusses.** A truss is a series of members taking stress in the direction of their length, placed together so as to form a triangle or system of triangles, which, when placed upon supports a certain distance apart, will, in addition to their own weight, sustain certain loads applied at the points where the members intersect. These points are called *panel points.*

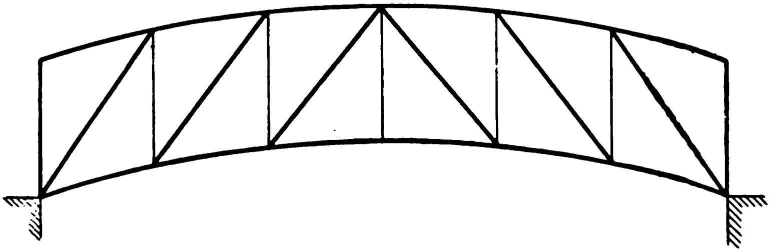


Fig. 3. Burr Truss Bridge, Arched.

5. **Bridge Trusses.** A bridge truss is one in which the members that carry the superimposed loads are in the same plane. Usually this plane is vertical.

6. **Truss Bridges.** A truss bridge is a structure consisting of two or more—usually two—bridge trusses connected by a system of beams called the *floor system,* which transfer to panel points the load for which the trusses are designed.

7. **Girders.** These are beams consisting of a wide, thin plate, called a *web plate,* with shapes, usually angles and narrow, thin plates called *flanges,* at the top and bottom edges. All are firmly riveted together. (See Part IV, "Steel Construction.")

8. **Girder Bridges.** These consist of usually two, sometimes

three, girders connected as in the case of truss bridges by a system of beams.

9. **Deck Bridges.** In cases where the floor system connects the trusses at their tops, the bridge is called a *deck bridge,* since the traffic moves on a *deck,* as it were (see Fig. 4).

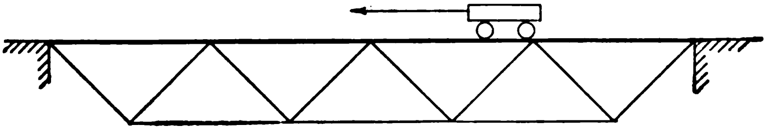


Fig. 4. Deck Bridge.

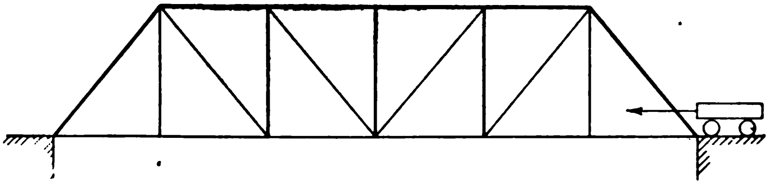


Fig. 5. Through Bridge.

10. **Through Bridges.** In cases where the floor system connects the bottoms of the trusses, the bridge is called a *through bridge,* as the traffic moves *through* the space between the trusses (see Fig. 5).

11. **Members of a Truss.** Each truss consists of a *top* and *bottom chord, end-posts,* and *web members.* The web members are further divided into *hip verticals, intermediate posts,* and *diagonals.* Fig. 6 shows these various classes, *A-A* being top chord, *B-B* bottom chord, *A-B* end-posts, vertical members *C-b* intermediate posts, *A-a* hip verticals, and *A-b* and *C-b* diagonals.

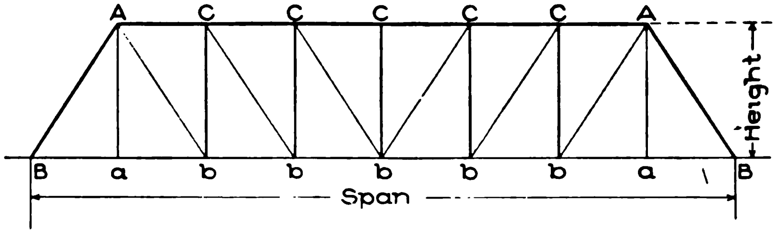


Fig. 6. Showing the Members of a Truss.

12. **Pony-Truss Bridges.** When the height of the trusses of a through bridge is less than the height of the loads that go over them, they are called *pony trusses,* and the bridge a *pony-truss bridge.*

13. **Lateral Bracing.** In all deck bridges, and in all through bridges except pony-truss bridges, the chords which are not connected by the floor system are connected by a horizontal truss system called the *lateral bracing.* In all bridges the chords which are connected by the floor system are connected by a horizontal truss system, also called the *lateral bracing.* One of these systems is called the *top lateral system,* as it connects the top chords; and the other is called the *bottom lateral system,* as it connects the bottom chords (see Fig. 7).

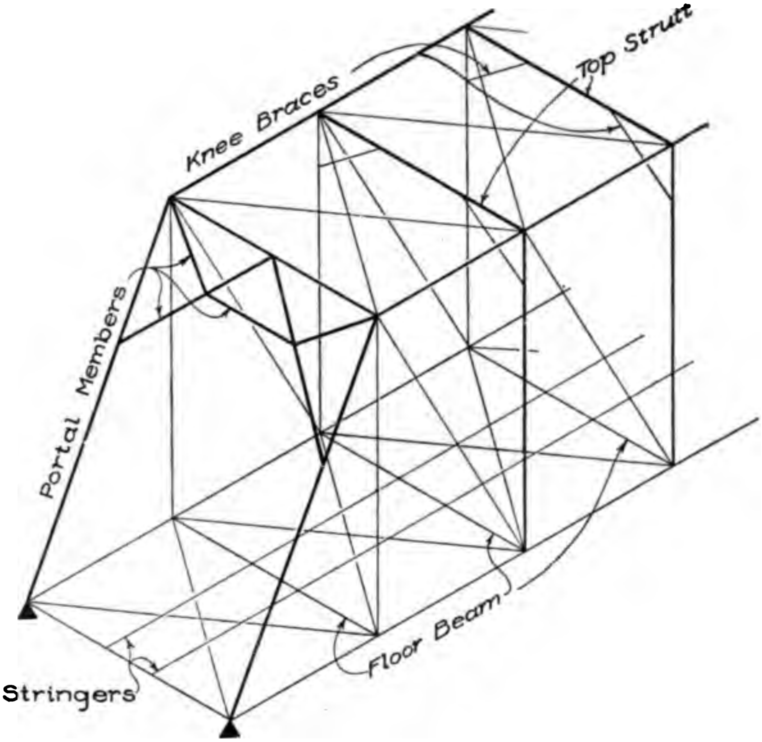
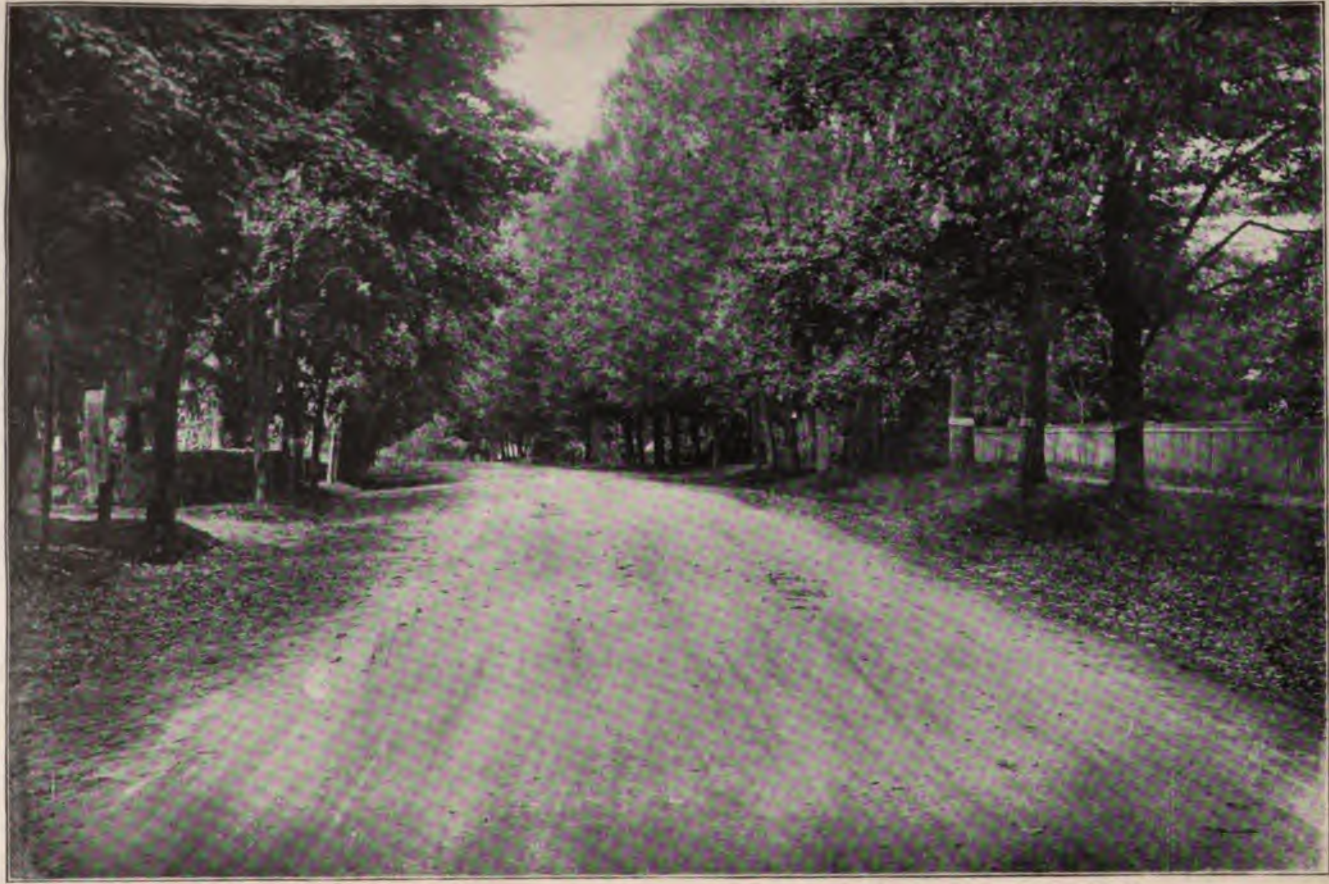


Fig. 7. Through Bridge, Showing Top and Bottom Systems of Lateral Bracing, also Portal Bracing and Floor System.

14. **Portals.** In through bridges, the end-posts of the pair of trusses are connected by a system of braces in order to preserve the rectangular cross-section of the bridge. This is called the *portal bracing* (see Fig. 7).

15. **Sway Bracing and Knee-Braces.** These serve the same purpose as the portal braces, and are either small struts or systems

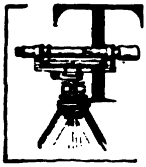


STATE HIGHWAY AT WESTON, MASSACHUSETTS

Showing road after tar treatment and after several weeks' use. Such a roadway is practically free from dust, and motor vehicles do not erode its surface so rapidly.

Courtesy of Massachusetts Highway Commission.

Foreword



THE marvelous developments of the present day in the field of Civil Engineering, as seen in the extension of railroad lines, the improvement of highways and waterways, the increasing application of steel and reinforced concrete to construction work, the development of water power and irrigation projects, etc., have created a distinct necessity for an authoritative work of general reference embodying the results and methods of the latest engineering achievement. The Cyclopedia of Civil Engineering is designed to fill this acknowledged need.

¶ The aim of the publishers has been to create a work which, while adequate to meet all demands of the technically trained expert, will appeal equally to the self-taught practical man, who, as a result of the unavoidable conditions of his environment, may be denied the advantages of training at a resident technical school. The Cyclopedia covers not only the fundamentals that underlie all civil engineering, but their application to all types of engineering problems; and, by placing the reader in direct contact with the experience of teachers fresh from practical work, furnishes him that adjustment to advanced modern needs and conditions which is a necessity even to the technical graduate.

chords are parallel. When such is the case, the stresses increase from the end toward the center, and there is a considerable difference between any two adjacent panels of the same chord. This necessitates different areas for each section. When the chords are not parallel, as in the bowstring truss, the stresses in the chords are so nearly equal that the same area is used throughout or nearly throughout the entire chord. Also, the stresses in the diagonals are nearly equal. These conditions would seem to indicate that this was a very economical form of truss. Theoretically it is; but practical considerations—such as the beveled joints and the posts which must be constructed to withstand reversals of stress—customarily limit this type to the longer spans.

19. **Web Characteristics.** The web systems of the Burr, Warren, Howe, Pratt, and Bowstring trusses are called *single* systems; that of the Whipple truss is a *multiple* system; while those of the Baltimore trusses are examples of webbing with *sub-systems.* As the maximum economical panel length has been found to be about 25 feet, which makes the economical height of the truss about 30 feet, and as the length of the span should not be more than ten times the depth, the span for trusses with simple systems of webbing is limited to about 300 feet. In order to increase the limiting span, multiple systems like that of the Whipple or similar ones were introduced. Calculations of stresses in members of the Whipple truss are somewhat unreliable on account of the fact that we are unable to tell just how the effects of the loads are distributed. For this reason, that type has gone out of use, and the sub-systems are used instead. These allow spans of twice the above limit; and, indeed, trusses with this type of webbing have been built up to and over 600 feet. This style of webbing can be applied to the bowstring truss, almost all long-span bridges being of this type with sub-systems of webbing.

WEIGHTS OF BRIDGES

20. **Formulæ.** In order to obtain the stresses due to the weight of the structure, the latter quantity must be known. As this weight can be determined only after the structure has been designed, it is evident that an assumption as to the weight must be made. The best method is to use the actual weight of a similar structure of like span which has been built. As the necessary data

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In order to give an idea of the relative weights of steel in different classes of bridges, let it be required to compute the dead weight of a 100-foot span of each class. For heaviest highway bridges to carry heavy interurban cars:

$$w = 600 + 180 + 27 \times 16 + \frac{16 \times 100}{12}\left(1 + \frac{100}{1\,000}\right) = 1\,358 \text{ lbs. per linear ft.}$$

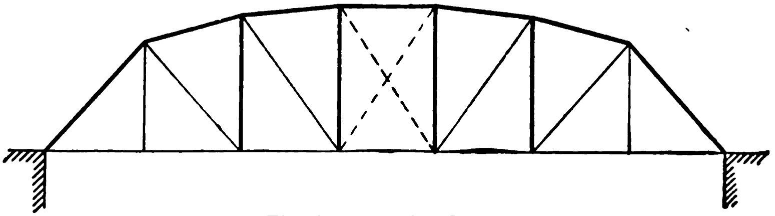
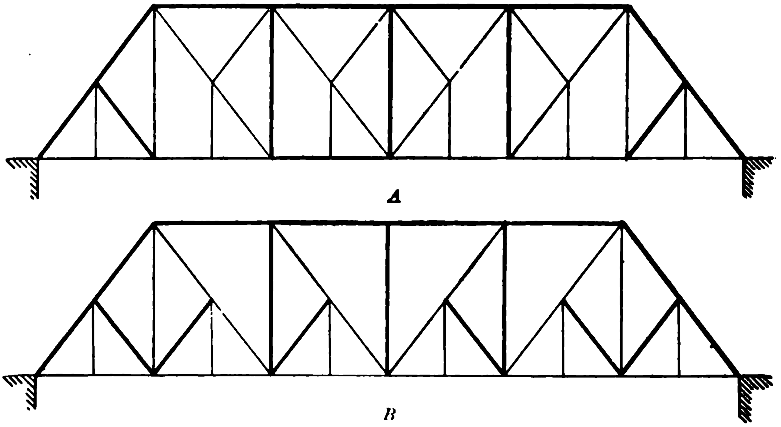


Fig. 13. Bowstring Truss.

For heavy riveted highway bridges to carry heavy farm engines:

$$w = 300 + 100 + 22 \times 16 + \frac{16 \times 100}{15}\left(1 + \frac{100}{1\,000}\right) = 870 \text{ lbs. per linear foot.}$$



A

B

Fig. 14. Two Forms of Baltimore Trusses.

For heavy pin-connected highway bridges to carry heavy farm or traction engines:

$$w = 34 + 22 \times 16 + 0.16 \times 16 \times 100 + 0.7 \times 100 = 710 \text{ lbs. per linear ft.}$$

For light country highway bridges to carry 100 pounds per square foot of floor surface:

$$w = 250 + 2.5 \times 100 = 500 \text{ lbs. per linear foot.}$$

If the total weight is required, the weight of the wooden floor must be added. Take, for example, the last bridge:

Weight of steel = 500 × 100 = 50 000 pounds.
" " floor = 100 × 16 × 10 = 16 000 pounds.
Total dead load = 66 000 pounds.

The weight per linear foot for a railroad truss bridge of 100-foot span is:

$$w = 650 + 7 \times 100 = 1\,350 \text{ lbs. per linear foot.}$$

This is about the same as that for a heavy interurban bridge The reason for this is that in addition to the heavy rolling stock of the electric road, the heavy highway traffic must be provided for. A deck girder of 100-foot span weighs:

$$w = 124 + 12 \times 100 = 1\,324 \text{ lbs. per linear foot.}$$

21. **Actual Weights of Railroad Spans.** In case actual weights can be obtained, a more exact analysis can be made. The weights of bridges indicated in the accompanying tables and diagrams, are based on actual constructions recently erected. These bridges represent the very best modern practice of engineers and manufacturers

The weights of through truss-spans made of medium steel and designed for E 50 loading, are given in Fig. 15. The weights include the weight of the ordinary open steel floor, and they also include the weight of the ties and rails, which is taken at 400 pounds per linear foot per track.

The weight of steel in medium steel deck plate-girder spans designed for E 50 loading, is given in Table III.

TABLE III

Weights of Deck Plate-Girders, Medium Steel
Loading E 50

Span (in feet)	Weight (in pounds)	Span (in feet)	Weight (in pounds)
15	5 300	70	59 500
20	7 800	75	67 300
25	11 800	80	76 300
30	14 500	85	94 200
35	18 800	90	105 500
40	23 300	95	114 200
45	27 400	100	123 600
50	32 400	105	146 000
55	38 800	110	161 700
60	45 500	115	174 900
65	51 500	120	187 000

The spans are the distance center to center of bearings; and the weights do not include the weight of the ties and rails, which is to be taken at 400 pounds per linear foot per track. Intermediate spans may be interpolated.

22. Actual Weights of Highway Spans. The actual weights of highway spans for heavy interurban trolley-cars and traffic, should preferably be computed from the formulæ of Shaw or Waddell (Table II). The weights of country bridges, including floor, may be taken from the diagram of Fig. 16.

LOADS

23. Classes of Loads. Those weights just given constitute what is called the *dead load* of the bridge. The traffic which passes over the bridge is called the *live* or *moving load.* In addition to the two classes mentioned, is the effect of the wind, which is designated as the *wind load.* These loads vary with the class of bridge, be it highway or railway, and with the purpose for which it is intended.

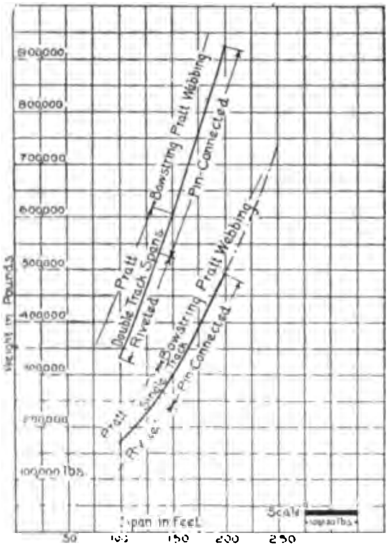


Fig. 15. Weights of Through Truss-Spans. Medium Steel, E 50 Loading.

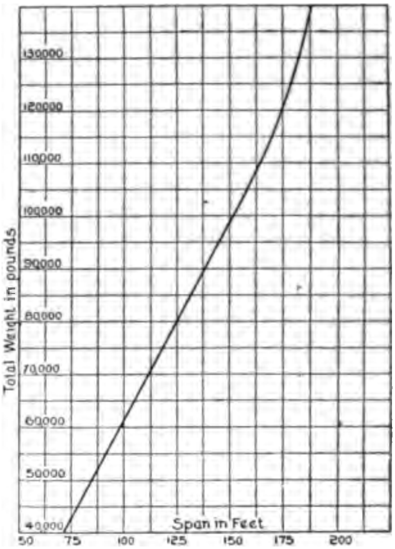


Fig. 16. Weights of Country Bridges, Including Floor.

24. Live Loads for Highway Bridges. Highway bridges are usually divided into several classes according to the traffic, which may be that of heavy interurban cars, light trolley-cars, farm engines, road rollers, teams, human beings, or some combination of these loadings. The standard specifications of J. A. L. Waddell or of Theodore Cooper are obtainable for a very small sum. Their purchase is advised, and the reader is referred to them for further information.

The trusses of country highway bridges are usually designed for a live load of 100 pounds per square foot of roadway. This may be considered good practice; and it is the law in some States. The floor system of these same bridges should be of sufficient strength to sustain 100 pounds per square foot of roadway, or a 12-ton farm engine having 4 tons on the two rear wheels, which are 12 inches wide and 6 feet apart, and 2 tons on each of the front wheels, which are 6 inches wide and 5 feet apart. The axles of this engine are spaced 8 feet center to center.

25. Live Loads for Railway Bridges. The loads for any particular railroad bridge are not always the same, on account of the great variation in the weights and wheel spacings of engines and cars. It is customary to design the bridge for the heaviest in use at the time of construction, or for the heaviest that could reasonably be expected to be built thereafter.

As the computations with engines were formerly somewhat laborious on account of the different weights and spacing of wheels, it has been proposed by some engineers to use a uniform load, called the *equivalent load*, which would give stresses the same, or very nearly the same, as those obtained by the use of engine loads. However, as these loads are different for each weight of engine, and also different for the chord members, the web members, and the floor-beam reaction of each different length of span, and as the labor of the computations, using engine-wheel loads, has been greatly reduced by means of diagrams, it does not seem as if this method would ever come into very general favor except for long-span bridges, where the live load is much smaller than the dead load.

The equivalent loads for Cooper's E 40 (see Fig. 85) are given in Table IV.

Most railways specify that their bridges shall be computed by using two engines and tenders followed by a train. The spacing of the wheels, and the load which comes on each wheel of the engines and tenders, are fixed by the railway company. The train is represented by a uniform load. Formerly there was a great diversity of practice among the different roads in regard to the engine and train loads specified; but practice has of late years become quite uniform,

TABLE IV

Equivalent Uniform Loads

Loading E 40

Span (in feet)	Equivalent Uniform Load			Span in feet	Equivalent Uniform Load		
	Chords	Web	Floor-Beam		Chords	Web	Floor-Beam
10	9 000	12 000	8 200	46	6 330	7 240	5 240
11	9 310	11 640	7 960	48	6 220	7 140	5 200
12	9 340	11 350	7 830	50	6 110	7 060	5 140
13	9 340	11 080	7 600	52	6 040	6 940	5 130
14	9 210	10 860	7 460	54	5 960	6 820	5 120
15	9 030	10 670	7 330	56	5 880	6 720	5 110
16	8 850	10 500	7 120	58	5 800	6 620	5 090
17	8 650	10 350	6 940	60	5 730	6 530	5 080
18	8 430	10 240	6 780	62	5 690	6 490	5 080
19	8 220	10 100	6 630	64	5 700	6 450	5 070
20	8 000	10 000	6 500	66	5 620	6 450	5 070
21	8 040	9 780	6 390	68	5 560	6 380	5 060
22	8 040	9 580	6 290	70	5 510	6 340	5 060
23	8 010	9 400	6 200	72	5 490	6 320	5 030
24	7 960	9 230	6 120	74	5 460	6 300	5 010
25	7 890	9 080	6 040	76	5 440	6 290	4 990
26	7 780	8 930	5 970	78	5 420	6 270	4 970
27	7 660	8 790	5 900	80	5 400	6 250	4 950
28	7 540	8 660	5 830	82	5 370	6 230	4 930
29	7 420	8 540	5 770	84	5 340	6 200	4 910
30	7 300	8 430	5 720	86	5 310	6 180	4 890
31	7 220	8 320	5 680	88	5 270	6 150	4 870
32	7 140	8 190	5 650	90	5 250	6 130	4 860
33	7 050	8 080	5 620	92	5 250	6 110	4 830
34	6 960	7 980	5 600	94	5 210	6 090	4 810
35	6 870	7 890	5 570	96	5 170	6 060	4 780
36	6 820	7 820	5 530	98	5 150	6 040	4 760
37	6 760	7 750	5 500	100	5 140	6 020	4 740
38	6 700	7 690	5 460	125	5 100	5 770	4 720
39	6 630	7 630	5 430	150	5 010	5 570	4 700
40	6 560	7 570	5 400	175	4 890	5 350	4 686
42	6 530	7 450	5 340	200	4 740	5 240	4 660
44	6 470	7 340	5 300	250	4 510	5 030	4 640

with an apparent tendency to standardize in accordance with the classes of loading specified by Cooper. Cooper's Class E 50, which represents the heaviest engines now in common use, was invented by Theodore Cooper, a consulting engineer of New York City. It is given in Fig. 17.

Lighter loadings for light traffic on the same general plan are advocated by Mr. Cooper, and are given at length in his "General Specifications for Iron and Steel Railway Bridges and Viaducts" (1906 edition).

26. Wind Loads. Some designers require that the stresses due to wind shall be computed by using 30 pounds per square foot of actual truss surface. This requires that you know the size of the members of the bridge before it is designed—which is evidently an impossibility; or that an assumption as to their size be made—which allows a chance for a mistake in judgment, especially in an inexperienced person. A more logical method, and one used to a great extent, is to assume a force of so many pounds per linear foot to act on the top and bottom chords and on the traffic as it moves across the bridge.

In highway through bridges, it is the usual practice to take the wind load as 150 pounds per linear foot of top and bottom chords, and 150 pounds per linear foot of the amount of live load which is on the bridge.

For railroad bridges, it is customary to use considerably higher values than those used in highway practice—not that the wind blows harder on railroad than on highway bridges, but so that the bracing designed by the use of these values may be sufficiently strong to stiffen the bridge not only against the wind, but also against the vibrations caused by the rapidly moving traffic. Good practice for through bridges is to use 150 pounds per linear foot of the top chord, 150 pounds per linear foot of the bottom chord, and 450 pounds per linear foot of live load on the bridge. This latter force is supposed to act at a line 8.5 feet above the base of the rail.

For deck bridges, for both highway and railway use, the unit-loads on the moving or live load remain the same, but the unit-loads on the top and bottom chords are reversed.

In computations involving the live load, it is always assumed that the live load moves over the bridge from right to left.

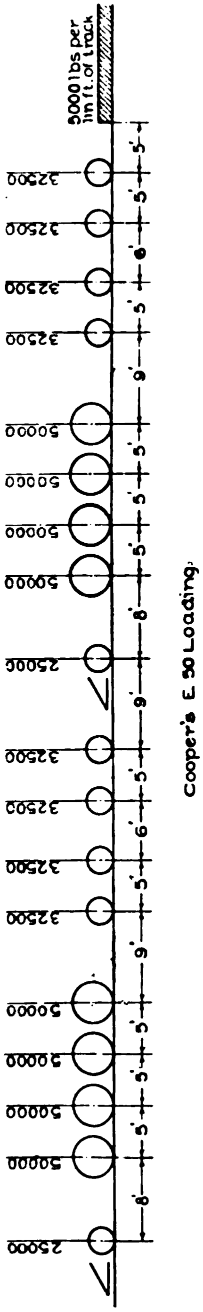


Fig. 17. Diagram of Cooper's E 50 Loading. Representing Heaviest Engines in Common Use.

THEORY

27. Principles of Analysis. The stresses in bridge trusses may be determined by both algebraic and graphic methods. In some cases, one is more expeditious than the other. Algebraic methods alone will be considered in this text.

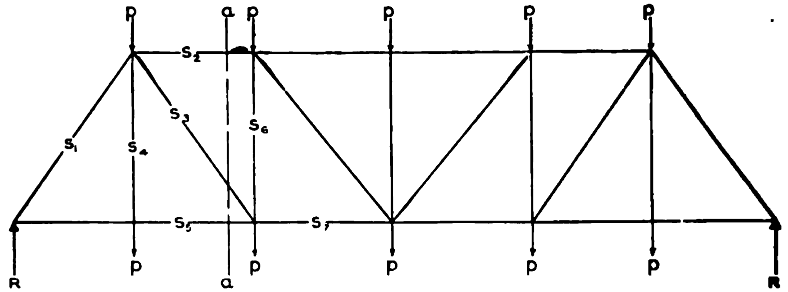


Fig. 18. Truss under Loads, Illustrating Principles of Analysis.

The analysis of stresses is based upon the fact that the interior stresses in a member or group of members hold in equilibrium the exterior forces. That this is a fact, can easily be understood. Consider a man pulling on a rope which is fastened at one end to an immovable object. There will be a stress in the rope equal to, and opposite in direction to, the pull exerted by the man. In order to prove this, cut the rope and apply a force equal and opposite to the pull exerted by the man, where the cut is made; and the rope and man will be in equilibrium. Also, suppose that a truss under loads, as indicated by the arrows, Fig. 18, were cut along the section *a-a*, and that forces F_2, F_3, F_5 equal to the stresses S_2, S_3, and S_5 were placed at the ends of the members as indicated in Fig. 19, then that portion of the truss to the left of the section would be in equilibrium. The interior stresses, represented by F_2, F_3, and F_5, would hold in equilibrium the exterior forces *p* and *R*.

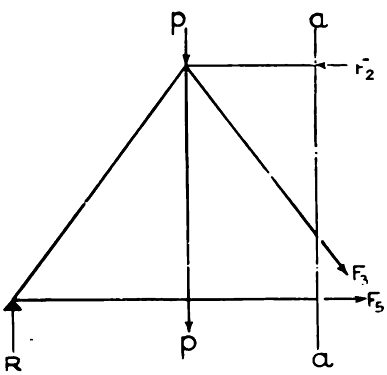
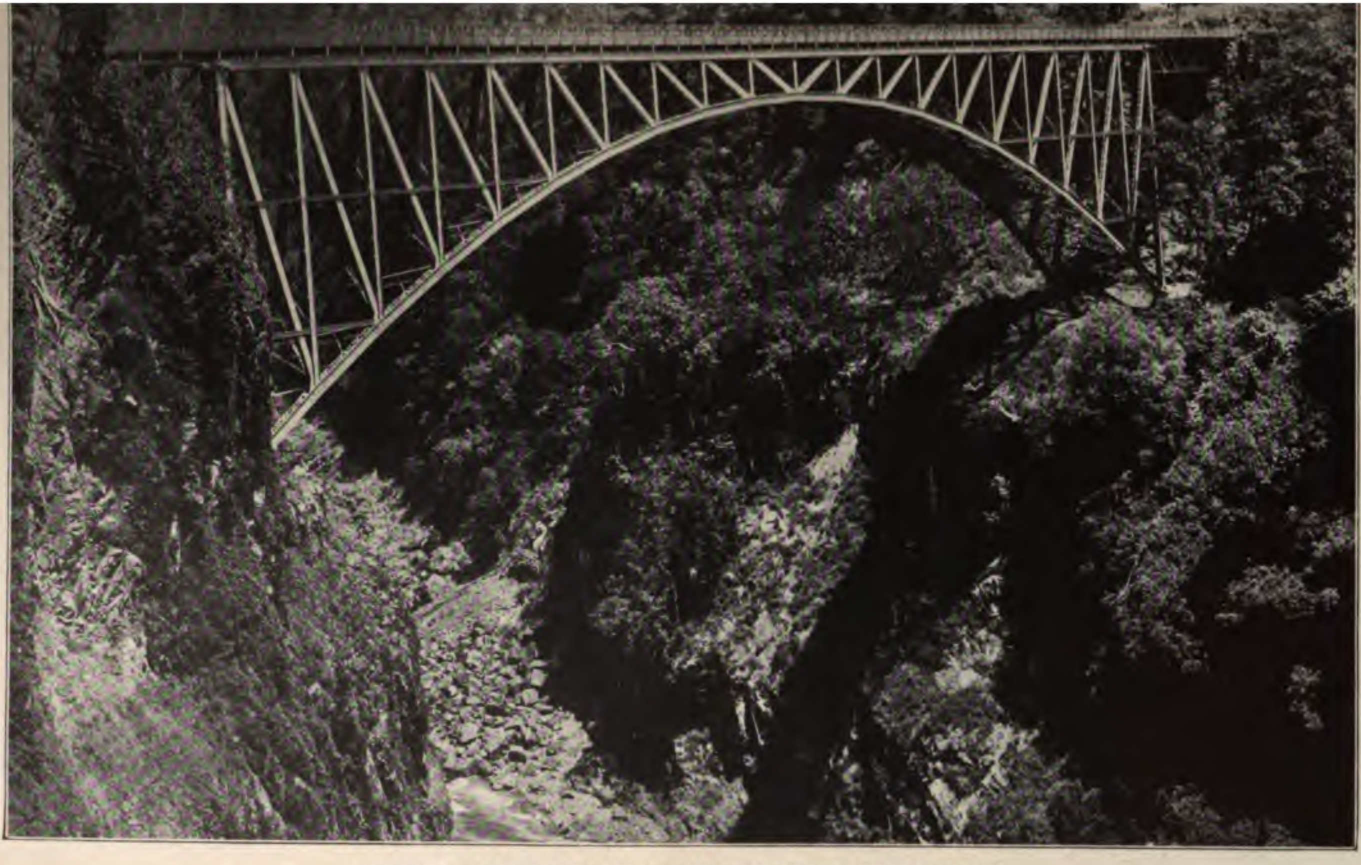


Fig. 19. Forces Substituted for Stresses in Truss of Fig. 18.



VICTORIA FALLS BRIDGE OVER ZAMBESI RIVER, SOUTH AFRICA

Completed in 1907. Of cantilever type; total length, 650 ft.; central span, 500 ft.; depth of gorge, over 400 ft. The bridge is about on a level with the crest of the Falls, which are a short distance upstream. The river, which is a mile wide at the Falls, is here confined within a very narrow gorge. A development of the 35,000,000 wasted horse-power has been projected, the power to be transmitted long distances for various industrial enterprises.

From inspection of Fig. 19, it will appear evident that, as the position of the truss to the left of the section is in equilibrium, the following statements are true:

1. The algebraic sum of the moments of the exterior forces and the stresses in the members cut by the section, is equal to zero. This is true of the moments taken about any or all points; for, if it were not, the portion of the truss would begin to rotate about some point, and would continue until equilibrium was established.

2. In a vertical plane, the algebraic sum of the components of the exterior forces and the stresses in the members cut by the section is equal to zero; for, if such were not the case, the portion of the truss shown would move up or down with a constant acceleration.

3. The algebraic sum of the components of the exterior forces and the stresses in the members cut by the section in a horizontal plane, is equal to zero; for, if such were not the case, the portion of the truss would move either to the right or to the left, with a constant acceleration.

4. From 2 and 3, above, it is evident that the algebraic sum of the components of the exterior forces and the stresses in the members cut by the section is equal to zero in any and all planes.

The section is not necessarily required to be a vertical line as in Fig. 19. It may be oblique, as in Fig. 20; or it may be a circular section, as shown in Fig. 20*a*. When the latter is the case, it is said that the sum of the components of the forces around the point U_1 is in equilibrium in any plane that may be taken.

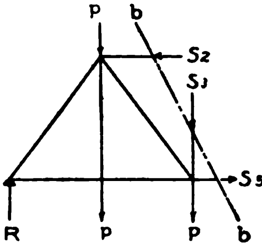


Fig. 20.
Oblique Section Cutting Truss.

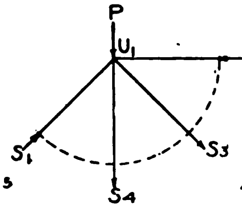


Fig. 20*a*.
Circular Section Cutting Truss.

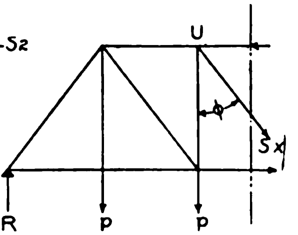


Fig. 21.
Illustrating Resolution of Forces.

It is also evident that the forces in the members cut by the section, and the exterior forces to the right, are in equilibrium. This condition is very seldom utilized in the determination of stresses, as that portion of the truss to the left of the section is almost always considered.

28. **Resolution of Forces.** This method is one of the simplest

and at the same time least laborious. The forces are generally resolved into their horizontal and vertical components, or parallel and perpendicular to some member. In cases where two unknown stresses occur, two equations can usually be formed, and these solved.

It should be assumed that the unknown stress acts *away from* the section which cuts it. It will then solve out, with the proper sign indicating the character of the stress—that is, whether it is tensile or compressive. Tensile stresses are indicated by placing the plus (+) sign before them, while a minus (−) sign indicates compression.

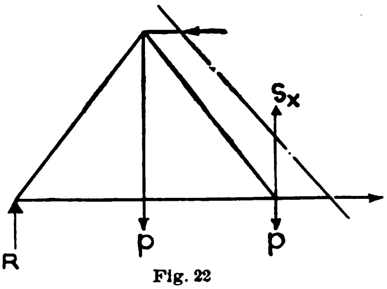


Fig. 22

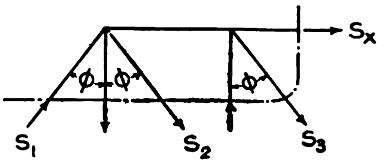


Fig. 23.

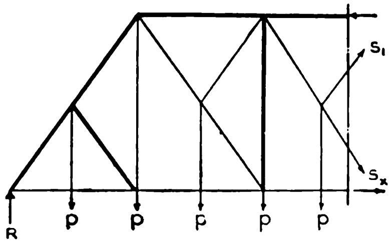


Fig. 24.

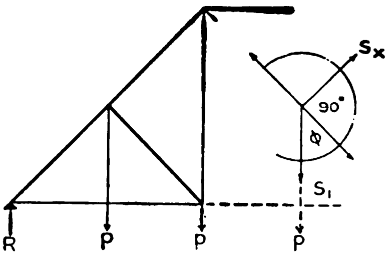


Fig. 25.

Diagrams Illustrating Application of Method of Resolution of Forces in Analysis of Trusses.

A few equations showing the application of the method of the resolution of forces can be written after inspection of Figs. 21 to 25 inclusive. In all cases, S_x is the unknown stress, and is assumed to be acting away from the section. The other stresses S_1, S_2, etc., are known, and their direction given them accordingly, it being toward the section if the member is in compression, and away from the section if the member is in tension. Forces or components acting upward or to the right are considered plus; those acting downward or to the left are considered minus. For a fuller explanation, see the instruction paper on Statics, Articles 17 to 23 inclusive.

In Fig 21, the sum of the vertical components is taken, and the equation is:

$$+R - p - p - S_x \cos\phi = 0;$$

whence,

$$S_x = -(+R - p - p)\sec\phi.$$

In Fig. 22, the section is oblique, and the sum of the vertical components is taken:

$$+R - p - p + S_x = 0;$$

whence.

$$S_x = -(+R - p - p).$$

In both of the above cases, it will be noted that the chord stresses do not enter into the equation, as their vertical components are zero.

In Fig. 23, the sum of the horizontal forces is used in determining the stress S_x. Note that the exterior forces R and p do not enter the equation, as they are not to the left of the section, and also their horizontal components are zero.

$$+S_1\sin\phi + S_2\sin\phi + S_3\sin\phi + S_x = 0;$$

whence,

$$S_x = -(S_1 + S_2 + S_3)\sin\phi.$$

In Fig. 24, the sum of the vertical forces is again used. Here the section cuts the member with the known tensile stress S_1.

$$+R - p - p - p - p - p + S_1\cos\phi - S_x\cos\phi = 0;$$

whence,

$$S_x = +(R - 5p)\sec\phi + S_1.$$

In Fig. 25, use is made of the fact that the sum of the components of the forces about a point is zero when they are resolved in any plane. Here they will be resolved perpendicular to the diagonal.

$$-S_1\sin\phi + S_x = 0.$$

$$S_x = +S_1\sin\phi.$$

These are some of the most common cases which occur in the determination of stresses in simple trusses. In all cases, follow this method of procedure:

1. Pass a section cutting as few members as possible, one of which must be the one whose stress is desired.
2. The stress in all the members cut, with but one exception, must be known.
3. Write your equation, always placing it equal to zero.
4. Solve for your stress.

29. Method of Moments. The stresses in all members of a truss can be determined by this method. By section 1 of Art. 28, the point about which the moments are considered can be taken anywhere. Fig. 26 represents the point as taken somewhere outside of the truss at a distance a above the point U_1. The equation will then be:

$$-S_1 \times a - S_2 \times b - S_3 (a + h) + Rp \pm P_1 \times 0 + P_2 \times p = 0.$$

This involves three unknown quantities, and therefore two other points should be taken, and two more equations written. By the use of the three equations, the stresses can be determined.

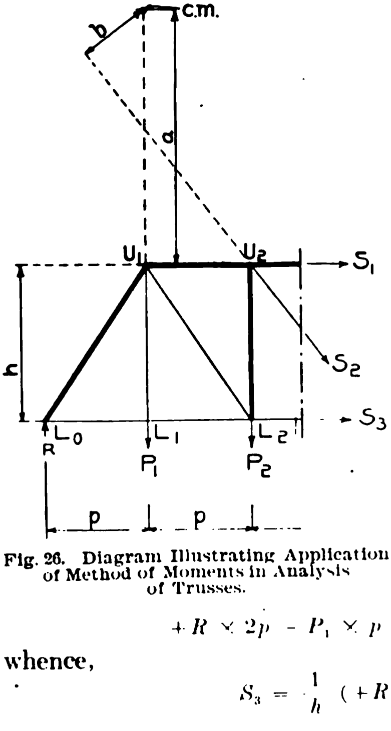


Fig. 26. Diagram Illustrating Application of Method of Moments in Analysis of Trusses.

It is customary to assume the center of moments at such a place that the moments of all the unknown stresses, with one exception, are zero. This condition requires that their lines of action pass through the center of moments. Let it be required to determine the stress S_3. If the center of moments is taken at U_2, then, as the lines of action of S_1 and S_2 pass through this point, their moments will be zero, and the following is true:

$$+R \times 2p - P_1 \times p \pm P_2 \times 0 - S_3 \times h = 0.$$

whence,

$$S_3 = \frac{1}{h} (+R \times 2p - P_1 \times p).$$

Likewise, if the top chord is curved, the center of moments can be taken in such a position that only the unknown stress will enter into the equation. If it is desired to determine the stress S_2, Fig. 27, the equation would be:

$$-S_2 \times l - R \times a + P_1 (a + p) + P_2 (a + 2p) = 0,$$

the center of moment being at O, the intersection of the lines of stress of S_1 and S_3. Solving the equation just stated,

$$S_2 = \frac{1}{l} \left\{ -Ra + P_1 (a + p) + P_2 (a + 2p) \right\}.$$

30. Stresses in Web Members. By reference to Articles 28 and 29, it is seen that several methods are presented for the solution of stresses in web members. Each should be adapted to the case in hand. The simplest method, and the one which is commonly used in all trusses with parallel chords, is by the resolution of the vertical forces. Fig. 21 is to be referred to. The equation given on page 19 is:

$$+R - p - p - S_x \cos \phi = 0.$$

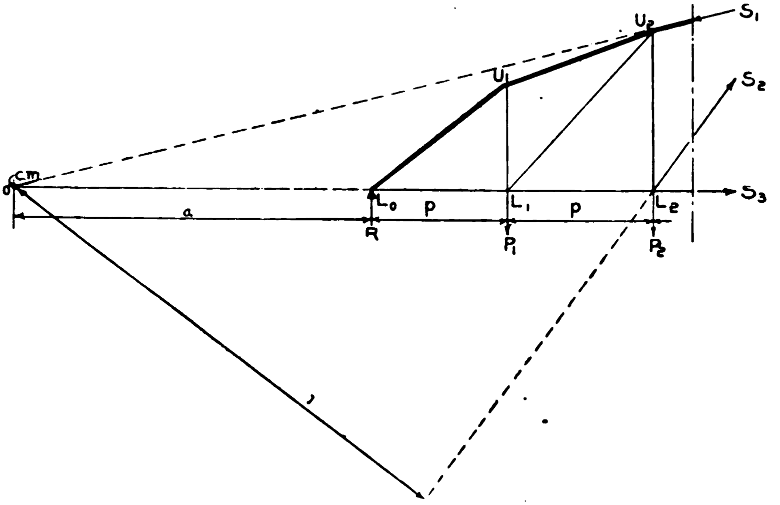


Fig. 27. Diagram Illustrating Application of Method of Moments in Analysis of Trusses. Top chord curved.

But $R - p - p$ is equal to V, the vertical shear at the section, and so the equation may now be written:

$$V - S_x \cos \phi = 0 \dots\dots\dots\dots\dots\dots (1)$$

whence the following important rule is deduced:

The algebraic sum of the vertical shear at the section and the vertical components of the stress in all of the members cut by the section, is equal to zero.

In trusses with horizontal chords and a simple system of webbing, the equation may be put in the form:

$$S_x = +V \sec \phi;$$

and the statement that the stress in any web member is equal to the shear *times* the secant of the angle that it makes with the vertical is true. The practice of using this latter statement is not to be encouraged, as it leads to confusion in the signs of the stresses. Equation (1) should be written in all cases, and the stress will then solve with its correct characteristic sign, indicating that the stress is either tensile or compressive.

As an example, let it be required to determine the stresses in the web members S_2 and S_3 of the Pratt truss shown in Fig. 28, the loads being in thousands of pounds. First, a section should be passed, cutting that member and as few others as possible. Next, the shear at that section should be computed. Then the vertical components of all the stresses cut by the section, and the vertical shear, should be equated to zero. Finally, solve the equation. Remember that the unknown stress is to be assumed as acting away from the section, and that forces or resultants acting downward are considered negative, while those acting upward are considered positive.

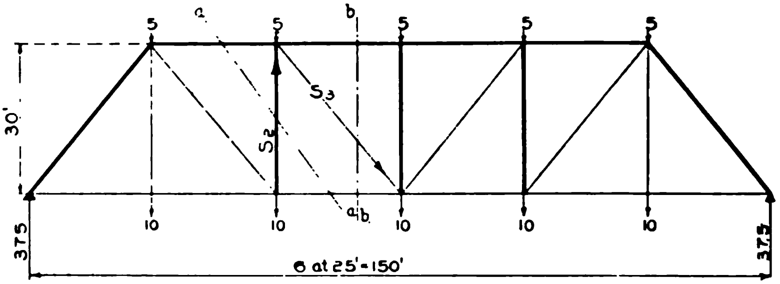


Fig. 28. Calculation of Stresses in Web Members of Pratt Truss.

To determine S_2:

The vertical shear at the section $a-a$ is:

$$+37.5 - 2 \times 10 - 5 = +12.5.$$

As the chord stresses do not exert a vertical component, the equation is:

$$+12.5 + S_2 = 0$$

$$S_2 = -12.5, \text{ which is a compressive stress of 12,500 pounds.}$$

Note that in this case the angle which the member makes with the vertical is zero, and the cosine and secant are unity.

To determine S_3:

The vertical shear at the section $b-b$ is

$$+37.5 - 2 \times 10 - 2 \times 5 = +7.5.$$

The equation is:

$$+7.5 - S_3 \cos \phi = 0$$

$$S_3 = +7.5 \sec \phi.$$

Sec ϕ is equal to $\sqrt{30^2 + 25^2} \div 30$, which is equal to 1.302; and therefore,

$$\begin{aligned} S_3 &= +7.5 \times 1.302 \\ &= +9.765, \text{ which is a tensile stress of 9,765 pounds.} \end{aligned}$$

31. Stresses in Chord Members. The stresses in chord may be obtained by either the method of moments or the method of resolution of forces, this latter being usually the resolution of horizontal forces.

In accordance with the text of Article 29, the following rule may be stated with regard to the solution of stresses in chord members by the method of moments:

Pass a plane section cutting the member whose stress is to be computed, and as few others as possible; then take the center of moments at such a point that the lines of action of as many forces as possible, the unknown one excepted, pass through that point; write an equation of the moments about this point of the known loads and forces to the left of the section, assuming the unknown force to act away from the section, and taking the known forces to act as given, the tensile stresses to act away from the section, and the compressive stresses to act towards the section; place the equation equal to zero, and solve.

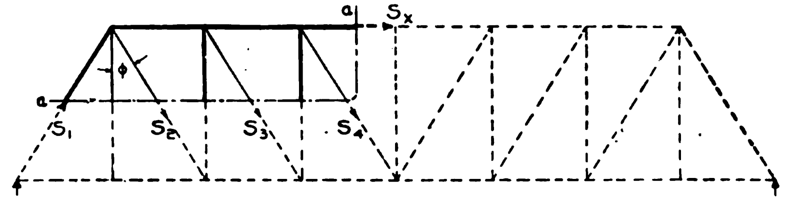


Fig. 29. Calculation of Stresses in Chord Members by "Tangent" or "Chord-Increment" Method.

The stress will solve out with its correct characteristic sign.

In the majority of cases a section can be made to cut three members only, one of the three being the one whose unknown stress is desired. In such cases, take the center of moments at the intersection of the other two, and proceed as before. As examples of this latter case, note the centers of moments at U_2, Fig. 26, and O, Fig. 27, and also the equations resulting therefrom.

When the method of resolution of forces is used, it is usually designated as the *tangent method* or the *chord increment method.* The simplest application of this method is to trusses with horizontal chords and vertical posts in the web members. Then the stress in any chord member is equal to the product of the sum of the shears in the panels up to that section, and the tangent of the angle which the diagonals make with the vertical.

This can readily be proved by reference to Fig. 29. Let it be required to determine the stress in the chord member S_x. Pass the section $a-a$. The stresses S_1, S_2, S_3, and S_4 are now computed, and are $S_1 = -V_1 \sec\phi$; $S_2 = +V_2 \sec\phi$; $S_3 = +V_3 \sec\phi$; and $S_4 = +V_4 \sec\phi$. Now noting the directions of the known stresses and assuming S_x to act away from the section, the equation of the horizontal component is:

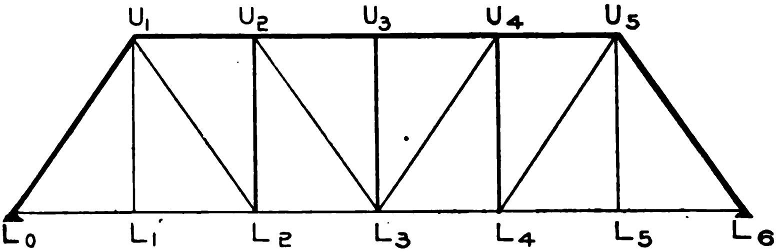


Fig. 30. Illustrating Method of Notation of Stresses and Members in a Through Bridge.

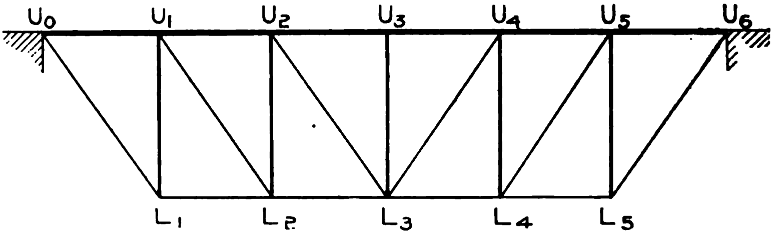


Fig. 31. Illustrating Method of Notation of Stresses and Members in a Deck Bridge.

$$+S_1 \sin\phi + S_2 \sin\phi + S_3 \sin\phi + S_4 \sin\phi + S_x = 0.$$

Now, substituting the values of S_1, S_2, etc., and remembering that $\sec\phi = \frac{1}{\cos\phi}$, the equation becomes:

$$V_1 \frac{\sin\phi}{\cos\phi} + V_2 \frac{\sin\phi}{\cos\phi} + V_3 \frac{\sin\phi}{\cos\phi} + V_4 \frac{\sin\phi}{\cos\phi} + S_x = 0,$$

from which,

$$S_x = -(V_1 + V_2 + V_3 + V_4) \tan\phi;$$
$$S_x = -\Sigma V \tan\phi.$$

From inspection of Fig. 29, it will be noticed that the stress in any section of the chord is equal to that in the section to the left of it, *plus* the increment (horizontal component) of the diagonal; hence the name *chord increment method.*

32. **Notation.** The practice hitherto used in designating stresses by S_1, S_2, etc., will now be discontinued, as it is inconvenient in the extreme; moreover, it is not the method used in practical work. The notation to be used is that given in Figs. 30 and 31, the former being for a through and the latter for a deck truss.

The practical advantages of this system are very great. When $U_1 U_2$ is noted, it is at once known to be the top chord of the second panel; $U_2 L_2$ is known to be the second vertical; while $U_2 L_3$ is at once recognized as the diagonal in the third panel. A stress in a member, as well as the member itself, is designated by the subscript letters at its ends. Thus $U_1 L_2$ may mean the member itself or the stress in the member. The text will clear this up. In analysis, the stress would be implied, while in design the member itself would be intended.

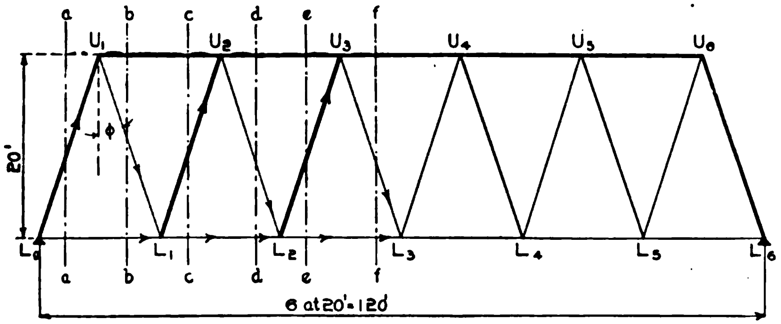


Fig. 32. Calculation of Stresses in a Six-Panel Warren Truss Through Bridge.

33. **Warren Truss under Dead Loads.** The Warren truss has its web members so built of angles and plates or of channels, that they can take either tension or compression. The top chord is of structural shapes, while the lower chord may be of built-up shapes or simply of bars.

Let it be required to determine all of the stresses in the six-panel truss of a through Warren highway 120-foot span for country traffic. The height is to be 20 feet. The outline is given in Fig. 32. According to Fig. 16, the total weight of the span, including wooden floor, is 76 000 pounds. Each truss carries one-half of this, or 76 000

÷ 2 = 38 000 pounds. As there are six panels, each panel load is 38 000 ÷ 6 = 6 333 pounds. This means that we must compute the stresses in the above truss by considering that a load of 6 333 pounds is at points L_1, L_2, L_3, L_4, and L_5. Of course there is some weight at L_0 and L_6; but this does not stress the bridge, as it is directed over the abutments or supports. The reactions at L_0 and L_6 are each equal to (5 × 6 333) ÷ 2 = 15 833 pounds.

The shears are next computed, and are:

$$V_1 = +15\,833 - 0 = +15\,833$$
$$V_2 = +15\,833 - 6\,333 = +9\,500$$
$$V_3 = +15\,833 - 2 \times 6\,333 = +3\,167$$

It is unnecessary to go past the center of the bridge, as it is symmetrical. The V_1 represents the shear on any section between L_0 and L_1; V_2 represents the shear on any section between L_1 and L_2; and V_3 represents the shear on any section between L_2 and L_3. The secant of the angle ϕ is:

$$\left(\sqrt{(20^2 + 10)^2}\right) \div 20 = 1.12.$$

The stresses in the web members are computed as follows:

For L_0U_1. Pass section $a-a$. Assume stress acting away from the section, as shown. Then,

$$V_1 + L_0U_1 \cos \phi = 0;$$
$$L_0U_1 = -V_1 \sec \phi;$$
$$L_0U_1 = -15\,833 \times 1.12 = -17\,700 \text{ pounds},$$

which shows that L_0U_1 has a compressive stress of 17 700 pounds.

For U_1L_1. Pass section $b-b$. Assume stress acting away from the section, as shown. Then,

$$V_1 - U_1L_1 \cos \phi = 0;$$
$$U_1L_1 = +V_1 \sec \phi;$$
$$U_1L_1 = +15\,833 \times 1.12 = +17\,700 \text{ pounds},$$

which shows that U_1L_1 has a tensile stress of 17 700 pounds.

For L_1U_2. Pass section $c-c$. Then,

$$V_2 + L_1U_2 \cos \phi = 0;$$
$$L_1U_2 = -V_2 \sec \phi;$$
$$L_1U_2 = -9\,500 \times 1.12 = -10\,640 \text{ pounds}.$$

For U_2L_2. Pass section $d-d$. Then,

$$+9\,500 - U_2L_2 \cos \phi = 0;$$
$$U_2L_2 = +9\,500 \times 1.12 = +10\,640.$$

For L_2U_3. Pass section *e* – *e*. Then,

$$+3\ 167 + L_2U_3 \cos \phi = 0;$$
$$L_2U_3 = -3\ 167 \times 1.12 = -3\ 540.$$

For U_3L_3. Pass section *f* – *f*. Then,

$$+3\ 167 - U_3L_3 \cos \phi = 0;$$
$$U_3L_3 = +3\ 167 \times 1.12 = +3\ 540.$$

The computation of the stresses in the chords is made by the method of moments, and is as follows:

For L_0L_1. Section *b* – *b* cuts U_1L_1 and U_1U_2, besides the member whose stress is desired, and therefore the center of moments will be taken at their intersection U_1. The equation is:

$$+15\ 833 \times 10 - L_0L_1 \times 20 = 0,$$

whence,

$$L_0L_1 = (+15\ 833 \times 10) \div 20;$$
$$= +7\ 917 = \text{a tension of } 7\ 917 \text{ pounds.}$$

For L_1L_2. Either section *c* – *c* or *d* – *d* may be used, and each shows the center of moments to be at U_2. The equation is:

$$+15\ 833 \times 30 - 6\ 333 \times 10 - L_1L_2 \times 20 = 0;$$
$$L_1L_2 = (+15\ 833 \times 30 - 6\ 333 \times 10) \div 20;$$
$$= +20\ 583 = \text{a tensile stress of } 20\ 583 \text{ pounds.}$$

For L_2L_3. Either section *e* – *e* or *f* – *f* may be used, and each shows the center of moments to be at U_3. The equation is:

$$+15\ 833 \times 50 - 6\ 333 \times 30 - 6\ 333 \times 10 - L_2L_3 \times 20 = 0;$$

whence,

$$L_2L_3 = +26\ 917.$$

The center of moments for U_1U_2 is at L_1; for U_2U_3, it is at L_2; and for U_3U_4, it is at L_3. The following equations can now be written:

$+20 \times U_1U_2 + 20 \times 15\ 833 = 0$; whence $U_1U_2 = -15\ 833$;
$+20 \times U_2U_3 + 40 \times 15\ 833 - 20 \times 6\ 333 = 0$; whence $U_2U_3 = -25\ 333$;
$+20 \times U_3U_4 + 60 \times 15\ 833 - 40 \times 6\ 333 - 20 \times 6\ 333 = 0$; whence $U_3U_4 = -28\ 500$.

A diagram of half of the truss should now be made, and all the stresses placed upon it. The dimensions should also be put upon this diagram. The student should cultivate this habit, as it shows him at a glance the general relation of stresses and the general rules of their variations. Fig. 33 gives the half-truss, together with the stresses and dimensions. The stresses in the members of the right half of the truss are the same as those in the corresponding members of the left half.

From inspection of the above diagram, it is seen that the chord stresses increase from the end toward the center; that the web stresses decrease from the end toward the center; and that all members slanting the same way as the end-post L_0U_1 have stresses of that sign, while all that slant a different way have an opposite sign. These relations are true of all trusses with parallel chords and simple systems of webbings.

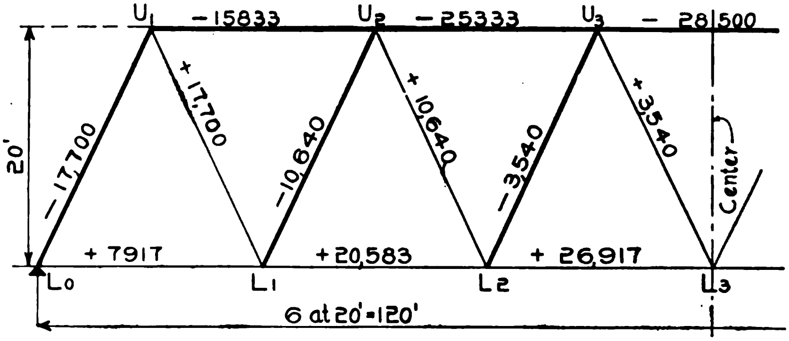


Fig. 33. Dimensions and Stress Diagram of Half a Six-Panel Warren Through Truss.

34. **Position of Live Load for Maximum Positive and Negative Shears.** The dead load, by reason of its nature, is an unchangeable load. The stresses due to it are the same at any and at all times.

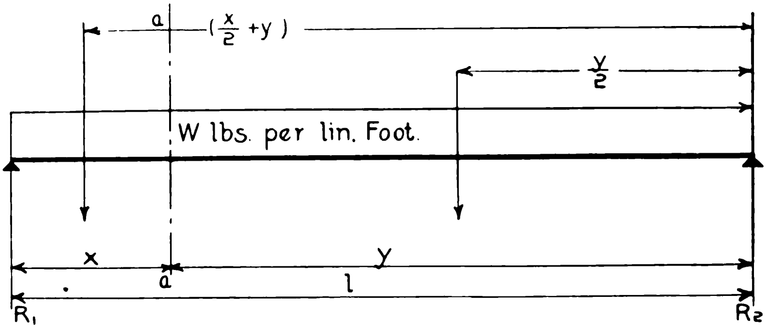


Fig. 34. Calculating Maximum Positive and Negative Shears in Simple Beam under Live Load. Conventional Method.

With the live load, the case is different. The live load represents the movement of traffic upon the bridge. At certain times there may be none on the bridge, while at other times it may fill the bridge partially or entirely. In such cases the shears due to live load will vary.

Conventional Method. It has been found that *the maximum positive shear at any section of a simple beam occurs when the beam is loaded from that section to the right support,* and that *the maximum negative shear occurs at the same section when this beam is loaded from the section to the left support.* This can be proved as follows:

Let a beam be as in Fig. 34, and let $a - a$ be the section under consideration. The reaction R_1 is due to the load wy on the part y, and to the load wx on the part x. That is,

$$R_1 = \left\{ wx\left(\frac{x}{2} + y\right) + wy\frac{y}{2} \right\} \div l.$$

Now the shear at the section $a - a$ is $R_1 - wx$; or,

$$\left\{ wx\left(\frac{x}{2} + y\right) + wy\frac{y}{2} \right\} \frac{1}{l} - wx = V_{a\,-\,a}$$

$$wx\left(\frac{\frac{x}{2} - y}{l}\right) + \frac{wy^2}{2l} - wx = V_{a\,-\,a}$$

$$\left\{ wx\left(\frac{\frac{x}{2} - y}{l}\right) - wx \right\} + \frac{wy^2}{2l} = V_{a\,-\,a}$$

From inspection of this last equation, it is seen that $wx\left(\frac{\frac{x}{2} - y}{l}\right)$ is the amount that is added to the reaction by loading the part x. Also, that $\frac{\frac{x}{2} - y}{l}$ is less than unity, is evident. The amount in brackets in the last equation represents the effect of the loading of the segment x of the beam. As this is negative and will only reduce the positive valued term $\frac{wy^2}{2l}$, it is therefore proved that to get the largest positive shear the beam should be loaded from the section to the right support.

From further inspection of the equation, it will be seen that the term in brackets, which represents the effect of the load on the segment x on the shear, is always negative; and that the term $\frac{wx^2}{2l}$, which represents the effect of the load on the segment y on the shear, is always positive. Hence, to get the largest negative shear at the section, the load should be on the segment x. That is, the loading should be from the section to the left support.

In a truss, the loads are placed at the panel points; and the above rules in application, should be formulated as follows:

To get the maximum positive shear at a section or in a panel, load all panel points to the right of it.

To get the maximum negative shear at a section or in a panel, load all panel points to the left of it.

Example. Determine the maximum positive and the maximum negative shears in the panels of the 7-panel Pratt truss shown in Fig. 35, the live panel load being 40 000 pounds. (It will be noticed that the height of the truss is not required.)

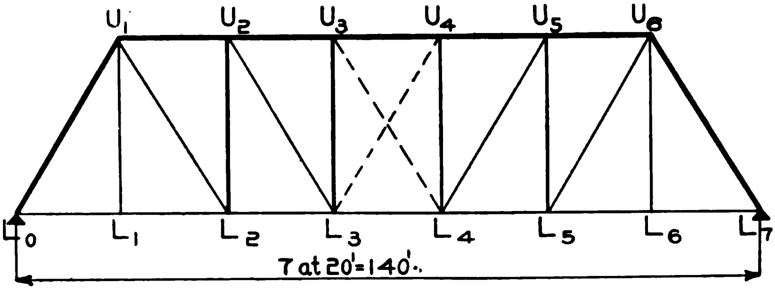


Fig. 35. Calculation of Shears in Panels of 7-Panel Pratt Truss.

For maximum +V in 1st panel, load L_1, L_2, L_3, L_4, L_5 and L_6.
" " +V " 2d " " L_2, L_3, L_4, L_5, and L_6.
" " +V " 3d " " L_3, L_4, L_5, and L_6.
" " +V " 4th " " L_4, L_5, and L_6.
" " +V " 5th " " L_5 and L_6
" " +V " 6th " " L_6.
" " +V " 7th " " no panel points at all.

The reaction produced by each of the loadings is equal to the shear for that particular case, since the shear at any section or in any panel is equal to the reaction *minus* the loads to the left of the section or panel, and, according to the method of loading, there are no loads to the left of the section; therefore the reaction is equal to the shear.

For the first panel, the computation is made as follows, the center of moments being, of course, at L_7:

$$(R_1 = +V_1) \times 7 \times 20 = 40 \times 20 + 40 \times 2 \times 20 + 40 \times 3 \times 20 + 40 \times 4 \times 20 + 40 \times 5 \times 20 + 40 \times 6 \times 20.$$

It will be seen that as 20 occurs in all terms of this equation, it can be factored out by dividing both sides by 20, and the result will be the same. The equation can now be written:

$$+V_1 \times 7 = 40 + 40 \times 2 + 40 \times 3 + 40 \times 4 + 40 \times 5 + 40 \times 6,$$

and can still be simplified by writing:

$$+V_1 = \frac{40}{7}(1 + 2 + 3 + 4 + 5 + 6) = +120.00,$$

which is the form customarily used, the panel length being taken as a unit of measurement. The other shears are now easily computed in a similar manner:

$$+V_2 = \frac{40}{7}(1 + 2 + 3 + 4 + 5) = +85.71$$

$$+V_3 = \frac{40}{7}(1 + 2 + 3 + 4) = +57.14$$

$$+V_4 = \frac{40}{7}(1 + 2 + 3) = +34.28$$

$$+V_5 = \frac{40}{7}(1 + 2) = +17.14$$

$$+V_6 = \frac{40}{7}(1) = +5.71$$

$$+V_7 = \frac{40}{7}(0) = +0$$

In computing the maximum negative shears, sometimes called the *minimum shears*, the reaction is not the same as the shear, as there are loads to the left of the section, and these must be subtracted. The loadings are:

For maximum $-V$ in 1st panel, load no points.
" " $-V$ " 2d " " L_1.
" " $-V$ " 3d " " L_1 and L_2.
" " $-V$ " 4th " " L_1, L_2, and L_3.
" " $-V$ " 5th " " L_1, L_2, L_3, and L_4.
" " $-V$ " 6th " " L_1, L_2, L_3, L_4, and L_5.
" " $-V$ " 7th " " L_1, L_2, L_3, L_4, L_5, and L_6.

It is evident that the maximum $-V_1$ is equal to zero, there being no loads on the span. The maximum negative shear in the second panel is equal to the reaction produced by loading the panel point L_1, and the load at L_1. Thus,

$$7R_1 = 40 \times 6$$

$$R_1 = \frac{40}{7}(6)$$

$$-V_2 = R_1 - \text{load at } L_1$$

$$= \frac{40}{7}(6) - 40$$

$$= -5.71$$

The other sheers are next computed as follows:

$$-V_3 = \frac{40}{7}(6 + 5) - 2 \times 40 = -17.14$$

$$-V_4 = \frac{40}{7}(6 + 5 + 4) - 3 \times 40 = -34.28$$

$$-V_5 = \frac{40}{7}(6 + 5 + 4 + 3) - 4 \times 40 = -57.14$$

$$-V_6 = \frac{40}{7}(6 + 5 + 4 + 3 + 2) - 5 \times 40 = -85.71$$

$$-V_7 = \frac{40}{7}(6 + 5 + 4 + 3 + 2 + 1) - 6 \times 40 = -120.00$$

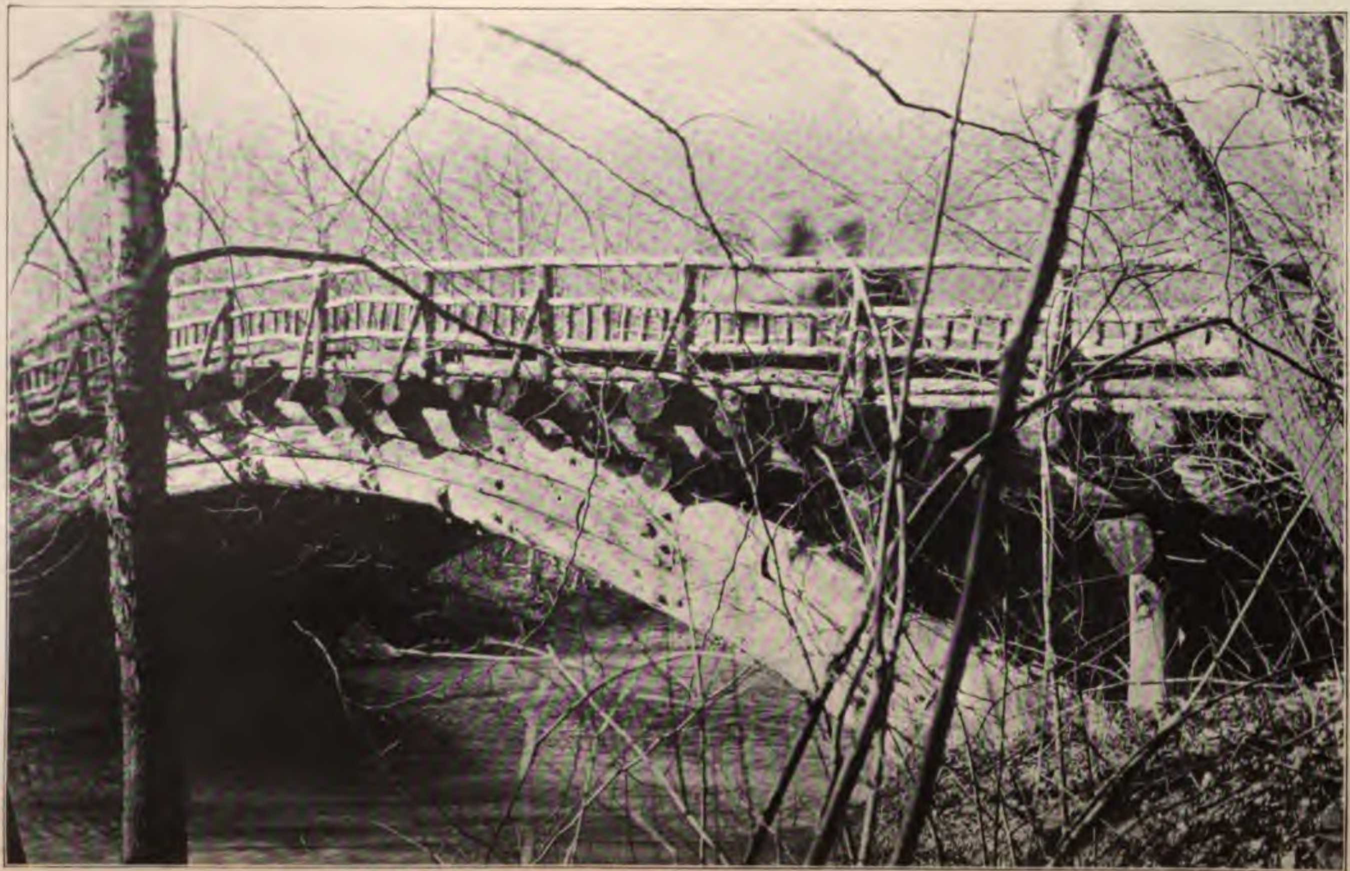
The maximum positive and the maximum negative live-load shears should now be written side by side, and inspected, in order to observe any existing relations which might help to lessen the labor of future computations. The values are given in thousands of pounds below:

LOCATION	MAX. + LIVE-LOAD SHEAR	MAX. – LIVE-LOAD SHEAR
V_1	+120.00	– 0.00
V_2	+85.71	– 5.71
V_3	+57.14	–17.14
V_4	+34.28	–34.28
V_5	+17.14	–57.14
V_6	+5.71	–85.71
V_7	+0.00	–120.00

It is at once seen that the negative shears are numerically equal in value to the positive ones, but that they occur in reverse order. This simplifies the labor required in the derivation of the negative shears; for, after computing the maximum positive shears, these may be written in reverse order, and the negative sign prefixed; the result will be the maximum negative shears.

The above method for maximum live-load shears is called the *conventional method*. It is the one that is almost universally used, and its use will be continued throughout this text.

Exact Method. On account of the fact that the floor stringers or joists transfer the loads to the panel points, it would be impossible to have a full panel live load at one panel point and no load at the panel point ahead or behind. In order to have a full panel load at one point, the stringers in the panels on both sides of the point must



AN UNUSUALLY FINE EXAMPLE OF RUSTIC BRIDGE BUILDING

In Rock Creek Park, Washington, D. C.

be full-loaded, and this would give a load at the panel point ahead, provided the bridge was fully loaded up to and not beyond the panel point ahead, equal in value to one-half of a full panel load (see Fig. 36). The uniform live load, in order to produce full panel loads at L_2, L_3 and L_4, will also produce one-half a panel load at L_1.

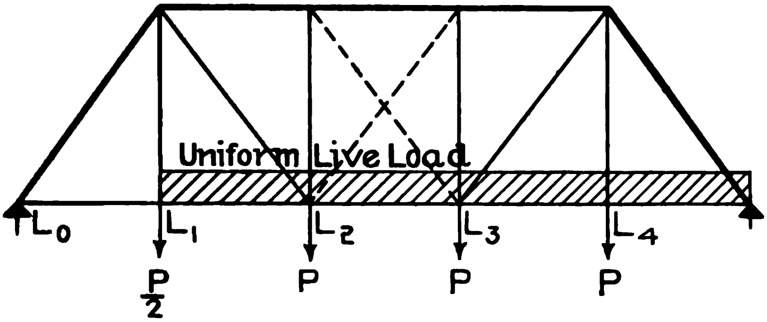


Fig. 36. Illustrating "Exact" Method of Calculating Live-Load Shears in Panels.

By the methods of differential calculus, it can be proved that the true maximum positive live-load shear occurs in a panel when the live load extends from the panel point to the right into that panel an amount (see Fig. 37) equal to

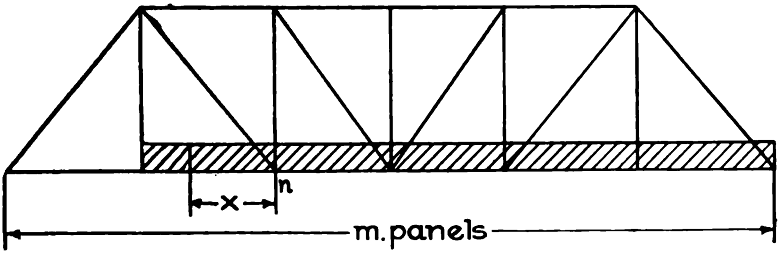


Fig. 37. Calculating Maximum Positive Live-Load Shear in Panel.

$$x = \frac{n}{m-1} p,$$

in which,

n = Number of the panel point to the left of the panel under consideration, counting from the right;
m = Total number of panels in the bridge;
p = Panel length.

Let the truss of Fig. 35 be considered, the live load being 2 000 pounds per linear foot of truss, and let it be required to determine the true maximum positive live-load shear in the 5th panel *from the right end.*

$$x = \frac{4}{7-1} \times 20 = 13.333 \text{ feet.}$$

There will now be $(4 \times 20 + 13.333) \times 2\,000 = 186\,666$ pounds on the truss; and the left reaction will be $\{186\,666 \times (4 \times 20 + 13.333) \div 2\} \div 140 = 62\,200$ pounds. From this must be subtracted the amount of the load on the 13.333 feet, which is transferred to the point L_2. This is equal to the reaction of a beam of a span equal to the panel length, loaded for a distance of 13.333 feet from the right support with a uniform load of 2 000 pounds per linear foot. This amounts to $(13.333 \times 2\,000 \times \frac{13.333}{2}) \div 20 = 8\,890$ pounds. The true shear is now:

$$+V_3 = +62\,200 - 8\,890 = +53\,310 \text{ pounds.}$$

The $+V_3$, as computed by the conventional method, was + 57 140, making a difference of 3 730 pounds between the two. If the true shears were computed and compared with the conventional shears, it would be found that the V_1 would be the same, and that the remainder of the conventional shears would be greater than the corresponding true shears. The difference between any two corresponding shears would increase from the left to the right end; that is, the difference between the conventional and exact shears would be greatest in the panel L_5L_6.

To get the maximum negative shear in any panel, load from the left support and out into the panel under consideration an amount $p - x$, and proceed in a manner similar to that above described.

As this method of exact or true shears is seldom employed, problems illustrating its application will here be omitted.

35. **Position of Live Load for Maximum Moments.** *In order to obtain the maximum moment at any point, the live load must cover the entire bridge.* Let the beam of Fig. 34 be considered, and let it be required to obtain the maximum moment at the section $a - a$. The reaction, as before computed, is:

$$R_1 = \left\{ wx\left(\frac{x}{2} + y\right) + \frac{wy^2}{2} \right\} \frac{1}{l},$$

all terms of which are positive. The moment at the section is:

$$M = R_1 \times x - wx\frac{x}{2};$$

and substituting for R_1 its value,

$$\begin{aligned} M &= \frac{wx^2}{l}\left(\frac{x}{2} + y\right) + \frac{wy^2x}{2l} - \frac{wx^2}{2}; \\ &= \frac{wx^2}{l}\left(\frac{x}{2} + y\right) - \frac{wx^2}{2} + \frac{wy^2x}{2l}; \\ &= wx^2\left(\frac{x}{2l} + \frac{y}{l} - \frac{1}{2}\right) + \frac{wy^2}{2l}. \end{aligned}$$

But $y = l - x$; therefore,

$$\begin{aligned} M &= wx^2\left(\frac{x}{2l} + \frac{l-x}{l} - \frac{1}{2}\right) + \frac{wy^2x}{2l}; \\ &= wx^2\left(\frac{2l-x}{2l} - \frac{1}{2}\right) + \frac{wy^2x}{2l}. \end{aligned}$$

The first term of this equation represents the effect of the load on the portion x, and the second term represents the effect of the load on the portion y. The value of M will always be positive. The quantity x varies between 0 and l. When $x = 0$, M is equal to 0. When $x = l$, the moment is equal to $+ wy^2x \div 2$. For all values of x between 0 and l, the first term is positive; and the second term being positive in all cases, it is therefore proved that for maximum live-load moments at any point, the entire span should be loaded, as loads on both segments add positive values to the moment value.

36. Warren Truss under Live Load. In order to analyze a truss intelligently, it is necessary to know its physical structure; that is, it must be known what character of stress can be withstood by the different members. The top chords of all trusses are built to take only compression, and the bottom chords are built to take only tension; while some web members of some trusses are built for tension stresses, some for compression stresses, and some for both. The characteristic of the Warren truss is that the web members are built so as to be able to withstand either tension or compression.

Let it be required to determine the live-load stresses in the Warren truss of Fig. 32. Let the live load per square foot of roadway, which is assumed to be 15 feet wide, be 100 pounds. The live panel load is then 100 × 15 × 20 ÷ 2 = 15 000 pounds, and the live-load reaction under full load is $2\frac{1}{2} \times 15\,000 = 37\,500$ pounds.

As the live load must cover the entire bridge to give maximum moments—and therefore maximum chord stresses, as the chord stress is equal to the moment divided by the height of the truss—a simple method for the determination of live-load chord stresses presents itself. The live load and the dead load being applied at the same points, and being different in intensity, the stresses produced will be proportional to the panel loads. The maximum live-load chord stresses (see Fig. 33) will then be equal to the dead-load chord stresses multiplied by 15 000 ÷ 6 333 = 2.371, and they are as follows:

$$\begin{aligned}
L_0U_1 &= -2.371 \times 17\,700 = -42\,000\\
U_1U_2 &= -2.371 \times 15\,833 = -37\,530\\
U_2U_3 &= -2.371 \times 25\,333 = -60\,050\\
U_3U_4 &= -2.371 \times 28\,500 = -67\,600\\
L_0L_1 &= +2.371 \times 7\,917 = +18\,770\\
L_1L_2 &= +2.371 \times 20\,583 = +48\,800\\
L_2L_3 &= +2.371 \times 26\,917 = +63\,850
\end{aligned}$$

The next step in order is to determine the maximum positive shears, and from these write the maximum negative shears. This is done as follows:

	+ Live-Load V	− Live-Load V
$V_1 = \frac{15\,000}{6}(1+2+3+4+5) =$	$+37\,500$	0
$V_2 = \frac{15\,000}{6}(1+2+3+4)$	$= +25\,000$	$-2\,500$
$V_3 = \frac{15\,000}{6}(1+2+3)$	$= +15\,000$	$-7\,500$
$V_4 = \frac{15\,000}{6}(1+2)$	$= +\ 7\,500$	$-15\,000$
$V_5 = \frac{15\,000}{6}$	$= +\ 2\,500$	$-25\,000$
$V_6 =$	$+\ \ 0$	$-37\,500$

The stresses produced by the positive shears are called the *maximum live-load stresses,* and are:

$+L_0U_1 \cos\phi + 37\,500 = 0 \qquad \therefore L_0U_1 = -37\,500 \times 1.12 = -42\,000$

$-U_1L_1 \cos\phi + 37\,500 = 0 \qquad \therefore U_1L_1 = +37\,500 \times 1.12 = +42\,000$

$+L_1U_2 \cos\phi + 25\,000 = 0 \qquad \therefore L_1U_2 = -25\,000 \times 1.12 = -28\,000$

$-U_2L_2 \cos\phi + 25\,000 = 0 \qquad \therefore U_2L_2 = +25\,000 \times 1.12 = +28\,000$

$+L_2U_3 \cos\phi + 15\,000 = 0 \qquad \therefore L_2U_3 = -15\,000 \times 1.12 = -16\,800$

$-U_3L_3 \cos\phi + 15\,000 = 0 \qquad \therefore U_3L_3 = +15\,000 \times 1.12 = +16\,800$

The stresses produced by the negative shears are called the ***minimum live-load stresses,*** and are:

$$\begin{array}{ll}
+L_0U_1 \cos\phi + 0 \quad = 0 & \therefore L_0U_1 = 0 \\
-U_1L_1 \cos\phi + 0 \quad = 0 & \therefore U_1L_1 = 0 \\
+L_1U_2 \cos\phi - 2\,500 = 0 & \therefore L_1U_2 = +2\,500 \times 1.12 = +2\,800 \\
-U_2L_2 \cos\phi - 2\,500 = 0 & \therefore U_2L_2 = -2\,500 \times 1.12 = -2\,800 \\
+L_2U_3 \cos\phi - 7\,500 = 0 & \therefore L_2U_3 = +7\,500 \times 1.12 = +8\,400 \\
-U_3L_3 \cos\phi - 7\,500 = 0 & \therefore U_3L_3 = -7\,500 \times 1.12 = -8\,400
\end{array}$$

These stresses, together with the dead-load stresses, should now be placed together as a half-diagram, as is done in Fig. 38, the stresses being rounded off to the nearest ten pounds and then expressed in thousands of pounds. No minimum live-load stress is given for the chords, as this will evidently be zero in all cases, since no position of the live load will cause a reversal of stress. It will be seen that the stresses produced by the negative shears are of opposite sign from the stress produced by the dead load, and these tend to decrease the dead-load stress by that amount; and in some cases (see L_2U_3 and U_3L_3, Fig. 38) it will be so large as to overcome the dead-load stress and therefore change the total stress from one kind to another. Do not forget, in considering any combination of the above stresses, that the dead load occurs with either the maximum or the minimum live load, but *not with both at the same time.*

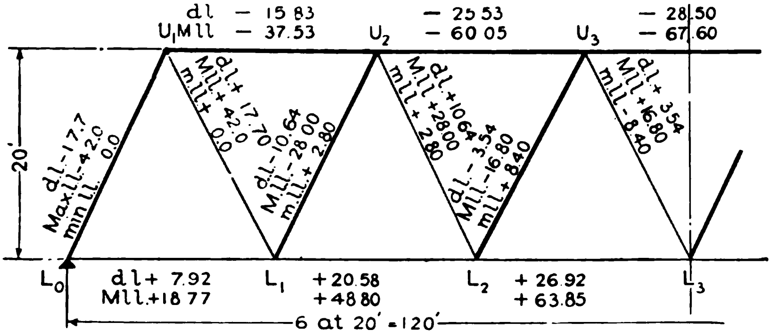
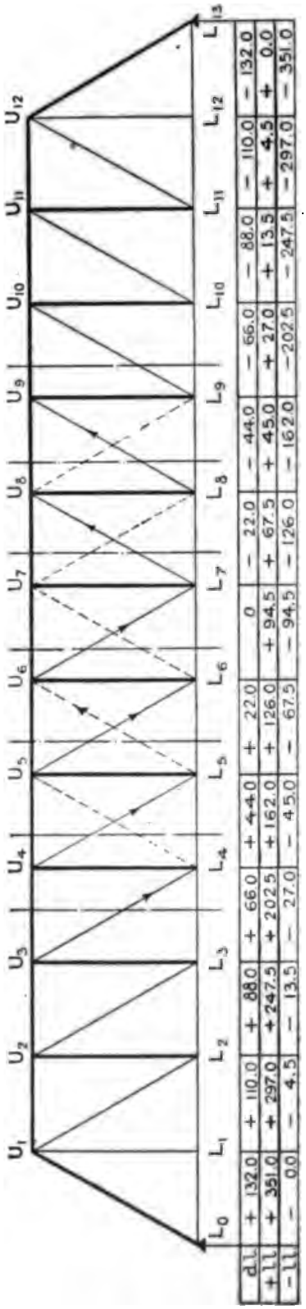


Fig. 38. Dimension and Stress Diagram of Warren Half-Truss under Live Load.

37. **Counters.** By reference to $U_3 L_3$ (Fig. 38), it is seen that when the live load is on the panel points L_1 and L_2 the total stress in the member is + 3.54 + (– 8.40) = –4.86, a compressive stress of 4 860 pounds; whereas, under dead load alone, the stress was + 3.54, a tensile stress of 3 540 pounds. If the member $U_3 L_3$ had been built of long, thin bars which could take only tension, and which consequently would have doubled up under the resultant compression



d.l.	+ 132.0	+ 110.0	+ 88.0	+ 66.0	+ 44.0	+ 22.0	0	− 22.0	− 44.0	− 66.0	− 88.0	− 110.0	− 132.0
+ l.l.	+ 351.0	+ 297.0	+ 247.5	+ 202.5	+ 162.0	+ 126.0	+ 94.5	+ 67.5	+ 45.0	+ 27.0	+ 13.5	+ 4.5	+ 0.0
− l.l.	− 0.0	− 4.5	− 13.5	− 27.0	− 45.0	− 67.5	− 94.5	− 126.0	− 162.0	− 202.5	− 247.5	− 297.0	− 351.0

Fig. 39. Load Diagram of 13-Panel Pratt Truss.

brought upon them by the combined action of the dead and minimum live-load stresses, then this member could not be used in this case, but some other arrangement would be necessary in order to insure the stability of the truss.

In the Warren truss, no special arrangement is necessary, as the web members are built so as to take either tension or compression; but with the Pratt and Howe trusses some special arrangement is necessary, as the diagonals are built to take one kind of stress only. The case of the Pratt will be considered first.

The Pratt truss has the diagonals made of long bars which take tension only, and the intermediate posts are constructed so as to be able to take compression only. It is not necessary to consider the intermediate posts, for the action of the web members is such that the resulting stresses are always compressive.

Let the 13-panel Pratt truss of Fig. 39 be considered. The panel length is 18 feet, the height 25 feet, the dead panel load 22 000 pounds, and the live panel load 58 500 pounds. The secant is $(18^2 + 25^2)^{\frac{1}{2}} \div 25 =$ 1.231. The dead-load shears and the maximum and minimum live-load shears are placed directly below their respective panels. Only those members are shown full-lined in Fig. 39 which act under the dead load. Note that the dead-load shears in the center panel being zero, the dead-load stress in the diagonals in the center panel would be $0 \times \sec \phi = 0$.

In the first four panels from either end, the live-load shear, which is of a different sign from that of the dead-load shear, is smaller than the dead-load shear, and therefore will not cause a reversal of stress in the member in that panel. For example, take $U_3 L_4$; then, for dead-load stress,

$$-U_3L_4 \cos \phi + 66.0 = 0 \qquad \therefore U_3L_3 = +66.0 \times 1.231 = +81.20$$

For live-load stress,

$$-U_3L_4 \cos \phi - 27.0 = 0 \qquad \therefore U_3L_3 = -27.0 \times 1.231 = -33.25$$

The total stress $= +81.20 - 33.25 = +47.95$, which is still tension.

Considering $L_9 U_{10}$, the stress equations are:
For dead-load stress,

$$+L_9U_{10} \cos \phi - 66.0 = 0 \qquad \therefore L_9U_{10} = +81.20$$

For live-load stress,

$$+L_9U_{10} \cos \phi + 27.0 = 0 \qquad \therefore L_9U_{10} = -33.25$$

The total stress, as before, is $+ 47.95$, or a tension of 47 950 pounds.

An inspection of the center panel and the two panels on each side of it, shows that the live-load shear is of a different sign from the dead-load shear, and is also greater in value than the dead-load shear. If the members shown in Fig. 39 were the only ones in the panels, then the dead-load stresses would be:

$$-U_4L_5 \cos \phi + 44.0 = 0 \qquad U_4L_5 = +54.20$$
$$-U_5L_6 \cos \phi + 22.0 = 0 \qquad U_5L_6 = +27.10$$
$$+L_7U_8 \cos \phi - 22.0 = 0 \qquad L_7U_8 = +27.10$$
$$+L_8U_9 \cos \phi - 44.0 = 0 \qquad L_8U_9 = +54.20$$

and the live-load stresses caused by the shear of opposite sign from that of the dead-load shear, are:

$$-U_4L_5 \cos \phi - 45.0 = 0 \qquad U_4L_5 = -55.40$$
$$-U_5L_6 \cos \phi - 67.5 = 0 \qquad U_5L_6 = -83.10$$
$$+L_7U_8 \cos \phi + 67.5 = 0 \qquad L_7U_8 = -83.10$$
$$+L_8U_9 \cos \phi + 45.0 = 0 \qquad L_8U_9 = -55.40$$

As no diagonal acts under dead load in the center panel, we may assume that $U_6 L_7$ acts under live load. The stresses which occur in this are:

$$+U_6L_7 \cos \phi + 94.5 = 0 \qquad U_6L_7 = +116.30$$
$$-U_6L_7 \cos \phi - 94.5 = 0 \qquad U_6L_7 = -116.30$$

The above shows that compressive stresses will occur in the diagonals which were built for tension only. These stresses are:

$$\begin{aligned}
U_4L_5 &= +54.20 - 55.40 = -1\,200 \text{ pounds} \\
U_5L_6 &= +27.10 - 83.10 = -56\,000 \text{ ''} \\
L_7U_8 &= +27.10 - 83.10 = -56\,000 \text{ ''} \\
L_8U_9 &= +54.20 - 55.40 = -1\,200 \text{ ''} \\
U_6L_7 &= 0 - 116.30 = -116\,300 \text{ ''}
\end{aligned}$$

If some provision were not made for these stresses, they would cause the members to crumple up and the truss to fail. In order to allow for them, diagonals are placed in the panels, as shown by the dotted and dashed lines. These members will take up the above stress; and moreover, as they slope the opposite way from the main members, they will be in tension.

In order to prove this, assume L_5U_6 to act when the live load is on points L_5, L_4, L_3, L_2, and L_1. Now, U_5L_6 will not be regarded, as its stress will be zero. Then the stresses will be:
For dead load,

$$+L_5U_6 \cos\phi + 22.0 = 0 \qquad L_5U_6 = -27.10.$$

For live load,

$$+L_5U_6 \cos\phi - 67.5 = 0 \qquad L_5U_6 = +83\ 10;$$

and the total stress in L_5U_6 will be $-27.10 + 83.10 = +56.00$.

In a similar manner, the stresses in the other members are: $L_4U_5 = +1.2$; $L_6U_7 = +116.30$; $U_7L_8 = +56.00$; and $U_8L_9 = +1.2$. These diagonals are called *counters* or *counter-bracing*.

From a consideration of the foregoing, it is evident that:

(*a*) *If the live-load shear in any panel is of opposite sign and greater than the dead-load shear in the same panel, then a counter is required.*

(*b*) *The stress in a counter is equal to the algebraic sum of the dead-load shear and the live-load shear of opposite sign times the secant of the angle it makes with the vertical.*

This is true for any truss with horizontal chords and a simple system of webbing with diagonals and verticals.

38. **Maximum and Minimum Stresses.** Some specifications require the member to be designed for the maximum stress, while others take into account the range of stress. In this latter case it is necessary to determine the minimum as well as the maximum stress. Except where a reversal of stress occurs—and this does not happen in trusses with horizontal chords—few specifications require any

but the maximum stresses to be computed. For that reason, little space will here be devoted to the minimum stresses, their computation in succeeding articles being thought to illustrate them sufficiently.

(*a*) *The maximum stress in a member is equal to the sum of the dead-load stress and the live-load stress of the same sign.*

(*b*) *The minimum stress is equal to the sum of the dead-load stress and the live-load stress of the opposite sign,* or *to the dead-load stress alone,* according to which gives the smallest value *algebraically.* By this latter statement it should be seen that if the maximum stress is -58.60, then 0 or $+18.00$ would be smaller than -3.00.

(*c*) It is evident that the minimum in all counters and in all main members in panels where counters are employed will be zero, for when the counter is acting the main member is not, and therefore its stress is zero. The reverse is also true.

(*d*) An exception to *a* is seen in the case of the counters. Here it is evident that the maximum stress is equal to the algebraic sum of the dead-load shear and the live-load shear of opposite sign *times* the secant of the angle which the counter makes with the vertical.

While it is true that in trusses with horizontal chords the loading for maximum shears will give the maximum live-load stress to be added to the dead load for *the maximum stress,* it is not always true that the loading for minimum live-load shears will give the stress to add to the dead-load stress to get the *minimum stress.* However, the loading for the minimum live-load shears will give the live-load stress to be added to the dead-load stress for the minimum stress, *except* in the case

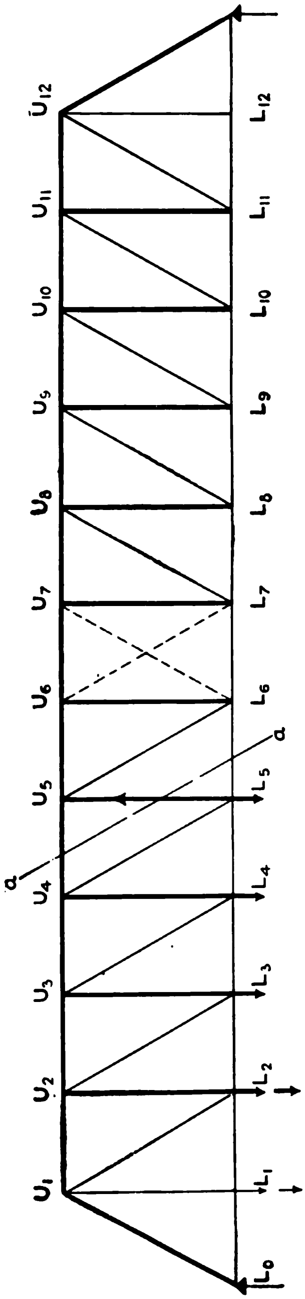
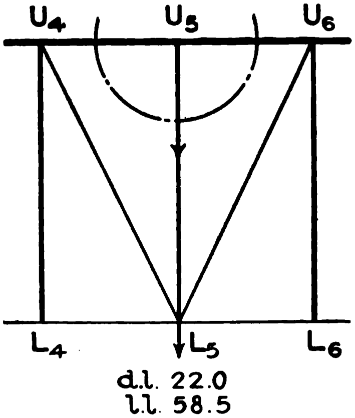


Fig. 40. Calculation of Stress in Vertical of 13-Panel Pratt Truss.

of verticals placed between panels each of which contains counters, and in that case it may or may not do so. In such cases a loading must be assumed—preferably the one for minimum shears—and the shears in the panels on each side of the vertical must be computed *for the loading assumed.* If the resultant shear is the same sign as the live load, then the main diagonal acts; if it is of different sign, then the counter acts.



d.l.v.	+ 44.0	+ 22.0
l.l.v.	− 9.0	− 67.5
Total v.	+	−

Fig. 41. Stress Diagram for Vertical in Truss of Fig. 40.

As an example, let it be required to find the minimum stress in the vertical U_5L_5 of the truss of Figs. 39 and 40. It is assumed that the loading for minimum shears will give the result. The section $a-a$ is then passed, and the live load placed on L_5 and all points to the left. The shears will then be as shown in Fig. 41. To obtain the shear in the panel L_4L_5, *under this loading,* it must be remembered that a load is at L_5; and so the shear is the shear in the panel L_5L_6 with the panel load at L_5 added, or, $-67.5 + 58.5 = -9.0$. The diagonals now act as indicated by Fig. 41, and the total stress in U_5L_5 is determined by passing a circular section around U_5, and it is :

$$-\text{Load at } U_5 - U_5L_5 = 0.$$

As there is no load at U_5, the stress in U_5L_5 is $= 0$. The same result will occur if points L_4 or L_3 and to the left are loaded; but if points L_2 and to the left are loaded, the members U_4L_5 and U_6L_5 will act, and the stress in U_5L_5 will then be equal to the shear on the section $a-a$. The stresses are: Dead-load, -22.0; and live-load, $+13.5$, which gives a total of -8.5; but as the maximum stress is $-22.0 - 126.0 = -148.0$, it is evident that 0 and not -8.5 is the minimum.

The computation of the maximum stress is as follows:

Load points L_6 and to the right. The shear on $a-a$ is, for dead load, +22.0; and for + live load, +126.0; and the equations of the stresses are:

$$
\begin{array}{ll}
+\ 22.0 + U_5L_5 = 0 & U_5L_5 = -\ 22.0 \\
+126.0 + U_5L_5 = 0 & U_5L_5 = -126.0 \\
\hline
 & \text{Max.} = -148\,0
\end{array}
$$

TRUSSES UNDER DEAD AND LIVE LOADS

39. The Pratt Truss. The Pratt truss is used to perhaps a greater extent than any other form; probably 90 per cent of all simple truss spans are of this kind.

Let it be desired to determine the stresses in the 8-panel 200-foot single-track span shown in Fig. 42, the height being 30 feet, the dead panel load being 30 000 pounds, and the live panel load 62 400 pounds. The secant is $\left(\overline{25}^2 + \overline{30}^2\right)^{\frac{1}{2}} \div 30 = 1.302$, and the cosine is 0.7685. The dead-load reaction is $3\frac{1}{2} \times 30.0 = 105.0$.

The dead-load shears are:

$$
\begin{aligned}
V_1 &= +105.0 \\
V_2 &= +\ 75.0 \\
V_3 &= +\ 45.0 \\
V_4 &= +\ 15.0 \\
V_5 &= -\ 15.0
\end{aligned}
$$

The dead-load chord stresses may be tabulated as follows (see Articles 27 and 29):

Dead-Load Chord Stresses

Member	Section	Center of Moments	Stress Equation	Stress
$L_0L_1 = L_1L_2$	$a-a$	U_1	$+\ 105.0 \times 25 - L_1L_2 \times 30 = 0$	$+\ 87.5$
L_2L_3	$b-b$	U_2	$+105.0 \times 50 - 30.0 \times 25 - L_2L_3 \times 30 = 0$	$+150.0$
L_3L_4	$c-c$	U_3	$+105.0 \times 75 - 30.0\,(25 + 50) - L_3L_4 \times 30 = 0$	$+187.5$
U_1U_2	$a-a$	L_2	$+105.0 \times 75 - 30.0 \times 25 + U_1U_2 \times 30 = 0$	-150.0
U_2U_3	$b-b$	L_3	$+105.0 \times\ 75 - 30.0\,(25 + 50) + U_2U_3 \times 30 = 0$	-187.5
U_3U_4	$c-c$	L_4	$+105.0 \times 100 - 30.0\,(25 + 50 + 75) + U_3U_4 \times 30 = 0$	-200.0

In determining dead-load stresses in web members, it is customary to assume one-third of the dead panel loads as applied at the upper chord points. This, as will be seen, makes no difference in the stresses in the chords or in the diagonals, the stresses in the verticals only being different from what is the case when all the dead load is taken on the lower chord.

The stresses in the diagonals (see Articles 27, 28, and 30) are:

Dead-Load Stresses in Diagonals

Member	Section	Shear on Section	Stress Equation,	Stress
L_0U_1	$o-o$	+105.0	$+105.0 + L_0U_1 \times 0.7685 = 0$	−136.70
U_1L_2	$a-a$	+ 75.0	$+ 75.0 - U_1L_2 \times 0.7685 = 0$	+ 97.60
U_2L_3	$b-b$	+ 45.0	$+ 45.0 - U_2L_3 \times 0.7685 = 0$	+ 58.60
U_3L_4	$c-c$	+ 15.0	$+ 15.0 - U_3L_4 \times 0.7685 = 0$	+ 19.53

In determining the stresses in the verticals, it is to be remembered that one-third the dead panel load (or 10.0) is at the panel points of the upper chord, and two-thirds (or 20.0) is at the lower chord. The stress in the hip vertical U_1L_1 is determined by passing a circular section around L_1. It is solved thus:

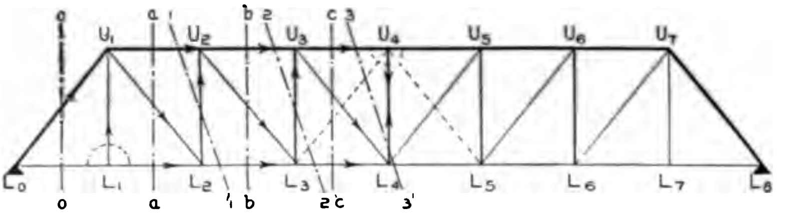


Fig. 42. Outline Diagram of 8-Panel Single-Track Pratt Truss Span.

$$-20.0 + U_1L_1 = 0 \qquad U_1L_1 = +20.0$$

In a similar manner the stress in U_4L_4 is found to be:

$$-10 - U_4L_4 = 0 \qquad U_4L_4 = -10.0$$

In order to find the stress in the remaining verticals, sections 1 − 1 and 2 − 2 are passed, cutting them, and the shears on these sections computed. The shears are:

$$V_{1-1} = +105.0 - 2 \times 20 - 1 \times 10 = +55.0$$
$$V_{2-2} = +105.0 - 3 \times 20 - 2 \times 10 = +25.0$$

The stress equations are written, remembering that as the verticals make an angle of zero with the vertical, their cosine is equal to unity. These equations are:

$$+U_2L_2 + 55.0 = 0 \qquad U_2L_2 = -55.0$$
$$+U_3L_3 + 25.0 = 0 \qquad U_3L_3 = -25.0$$

The live-load chord stresses will be proportional to the dead-load chord stresses, as both loads cover the entire truss in exactly the same manner. The ratio of the panel loads by which the dead-load chord stresses are multiplied in order to get the live-load chord stresses, is $\frac{62\ 400}{30\ 000} = 2.08$, and the chord stresses are:

$$L_0L_1 = L_1L_2 = +\ 87.5 \times 2.08 = +182.0$$
$$L_2L_3 = +150.0 \times 2.08 = +312.0$$
$$L_3L_4 = +187.5 \times 2.08 = +390.0$$
$$U_1U_2 = -150.0 \times 2.08 = 312.0$$
$$U_2U_3 = -187\ 5 \times 2.08 = -390.0$$
$$U_3U_4 = -200.0 \times 2.08 = -416.0$$

As the entire bridge is to be loaded to get the maximum stress in L_0U_1, it is therefore equal to the dead-load stress *times* the above ratio; or $L_0U_1 = -136.70 \times 2.08 = -284.20$.

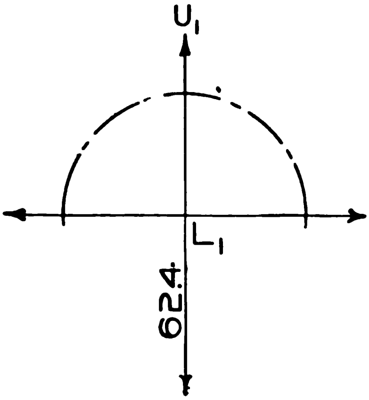


Fig. 43. Calculation of Maximum Live-Load Stress in a Vertical of Span of Fig. 42.

The maximum live-load stress in U_1L_1 is determined by passing a circular section around L_1, and is solved (see Fig. 43) from the equation:

$$+U_1L - 62.4 = 0 \quad \therefore U_1L_1 = +62.4$$

For U_1L_2, the section $a - a$ is passed, and the points L_2 and to the right are loaded. The maximum shear is:

$$+V_2 = +\frac{62.4}{8}(1 + 2 + 3 + 4 + 5 + 6) = +163.8;$$

and the stress equation is:

$$+163.8 - U_1L_2 \times 0.7685 = 0 \qquad \therefore U_1L_2 = +213.2.$$

In a similar manner, pass section $b - b$, and load points L_3 and to the right, and the shear and the stress equations for U_2L_3 are:

$$+V_3 = +\frac{62.4}{8}(1 + 2 + 3 + 4 + 5) = +117.0$$
$$+117.0 - U_2L_3 \times 0.7685 = 0 \qquad \therefore U_2L_3 = +152.4$$

For U_3L_4, the section $c - c$ is passed, and the panel points to the right are loaded. The shear and stress equations are:

$$+V_4 = +\frac{62.4}{8}(1 + 2 + 3 + 4) = +78.0$$

$$+78.0 - U_3L_4 \times 0.7685 = 0 \qquad \therefore U_3L_4 = +101.6$$

For the maximum stresses in the verticals, sections 1–1, 2–2, and 3–3 are passed, and in each case the panel points to the left of these loaded. The shears are

$$V_{1-1} = \frac{62.4}{8}(1 + 2 + 3 + 4 + 5) = +117.0$$

$$V_{2-2} = \frac{62.4}{8}(1 + 2 + 3 + 4) = +78.0$$

$$V_{3-3} = \frac{62.4}{8}(1 + 2 + 3) = +46.8$$

The stress equations for U_2L_2 and U_3L_3 are simple, as only three members are cut. They are:

$$+117.0 + U_2L_2 = 0 \qquad \therefore U_2L_2 = -117.0$$
$$+78.0 + U_3L_3 = 0 \qquad \therefore U_3L_3 = -78.0$$

It is seen that the section 3–3 cuts the member L_4U_5, and therefore the stress in this must be determined before the stress equation can be written, as its vertical component will enter into it. However, by comparing the dead-load shear in that panel, which is −15.0, and the live-load shear V_{3-3}, which is +46.8, it is seen that the resultant shear is +; and, as this is of opposite sign from the dead-load shear, a counter is required and is acting. The stress in U_5L_4 is zero, and the diagonals act as in Fig. 44, the section 3 – 3 then cutting three members. The stress equation is + 46.8 + U_4L_4 = 0, from which U_4L_4 = – 46.8.

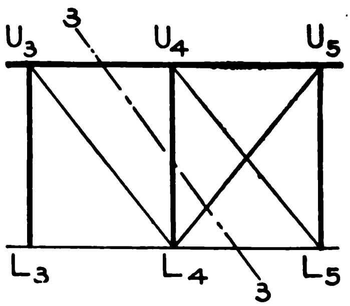


Fig. 44. Calculation of Stress in Diagonal of Span of Fig. 42.

Care should be taken not to add to this −46.8 the −10.0 derived as dead-load stress on page 44, in order to get the maximum stress, as the −10.0 previously derived was the dead-load stress in U_4L_4 when U_3L_4 and L_4U_5 were acting. The dead-load stress which goes with the live-load stress of −46.8 acts simultaneously with it, and is the dead-load stress in U_4L_4 when the members U_3L_4 and U_4L_5 are acting as in Fig. 44. The dead-load shear on the section 3 – 3 would then be the left reaction *minus* the loads at points U_1, U_2, U_3, L_1, L_2, L_3 and L_4; or,

$$V_{3-3} = +105.0 - 3 \times 10 - 4 \times 20 = -5.0;$$

and

$$-5.0 + U_4L_4 = 0 \qquad \therefore U_4L_4 = +5.0.$$

Remember that this +5.0 can act only when the live load tends to produce a stress of −46.8; and thus the total stress in U_4L_4 with live load in that position is −46.8 + 5.0 = −43.8, while with dead load only in the truss the stress is −10.0.

The dead-load shears and the maximum + and − live-load shears should now be written for inspection, in order to investigate for counters and then for the minimum stresses. Those whose derivation has not been given should be easily computed by the student at this time. The shears are

Dead-Load		+ Live-Load	− Live-Load
V_1	+105.0	+218.4	0
V_2	+ 75.0	+163.8	− 7.8
V_3	+ 45.0	+117.0	−23.4
V_4	+ 15.0	+ 78.0	−46.8

From a study of these it is seen that a counter is required in the 4th panel according to rule *a*, Article 37; and according to rule *b* of the same article, the maximum stress is (−46.8 + 15.0) × 1.302 = +41.4, the minimum stress for it and also U_3L_4 being zero according to the same article. A counter is also required in panel 5, as the truss is symmetrical.

The minimum live-load stress in U_1L_1 is zero, and occurs when no live load is at the point L_1.

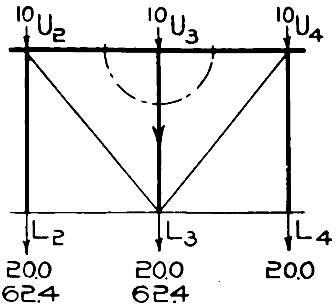
The minimum live-load stresses in the diagonals U_1L_2 and U_2L_3 occur when the truss is loaded successively to the left of the sections *a* – *a* and *b* – *b*, in which case the shears are −7.8 and −23.4 respectively. The stress equations are

$$-U_1L_2 - 7.8 \times 0.7685 = 0 \qquad \therefore U_1L_2 = -10.16$$
$$-U_2L_3 - 23.4 \times 0.7685 = 0 \qquad \therefore U_2L_3 = -29.15$$

The minimum live-load stress in U_2L_2 is obtained by passing section 1 — 1 and loading the panel points to the left. The live-load shear is the same at this section as it is at the section $b-b$ —namely, -23.4. The stress equation is

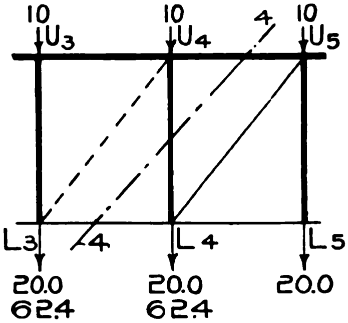
$$+U_2L_2 - 23.4 = 0 \qquad \therefore U_2L_2 = +23.4$$

To determine the minimum live-load stress in U_3L_3, proceed as indicated on page 42. By loading points L_3 and to the left, the live-load shear in the 4th panel will be -46.8, and in the 3d panel *under this same loading* it will be $-46.8 + 62.4 = +15.6$. The sign of the total shear in the two adjacent panels, and the members acting, are shown in Fig. 45. The stress in U_3L_3 is then determined by using a circular section around U_3, and is simply the dead load at U_3, there being no live-load stress in the member when the bridge is loaded as has been done.



d.l.v.	+45.0	+15.0
l.l.v.	+15.6	−46.8
Total v.	+	—

Fig. 45.

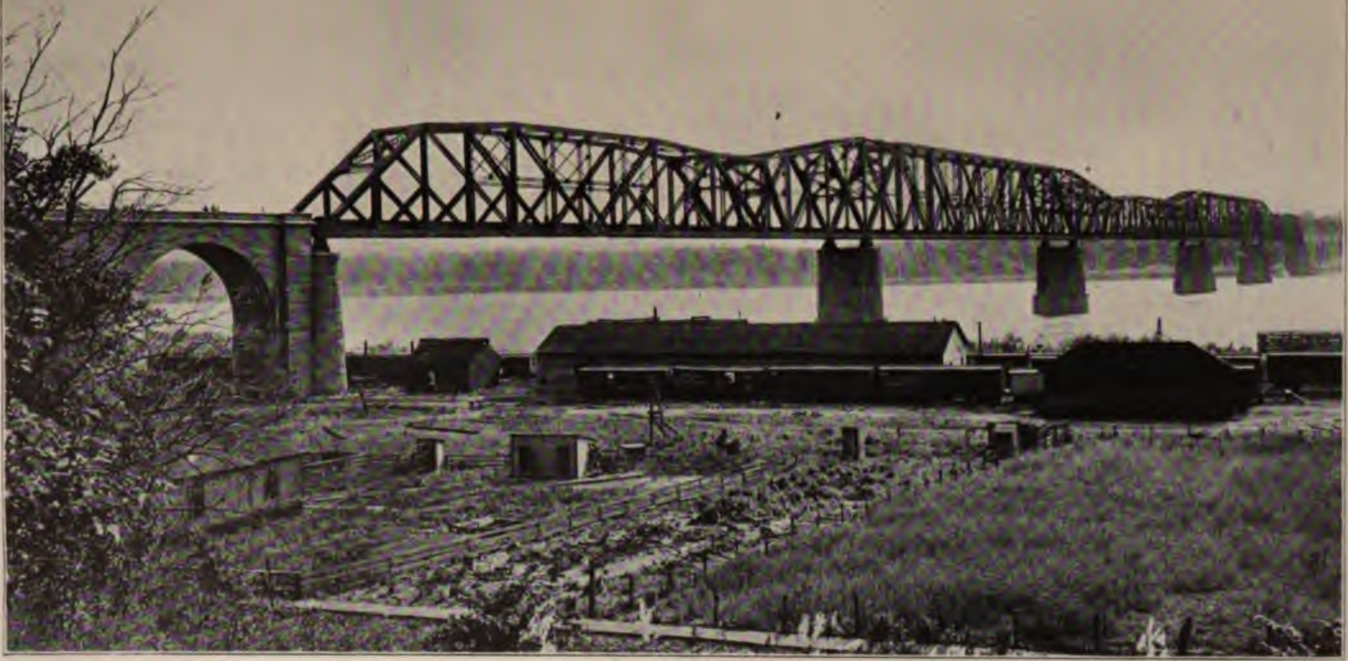


d.l.v.	+15.0	−15.0
l.l.v.	−15.6	−78.0
Total v.	—	—

Fig. 46.

Stress Diagrams for Verticals in Span of Fig. 42.

In finding the minimum live-load stress and also the minimum stress in U_4L_4, the same method of procedure will be followed. Let L_4 and to the left be loaded. Then the shear in the 5th panel is -78.0, and *under this same loading* the shear in the 4th panel is $-78.0 + 62.4 = -15.6$. The sign of the total shear in each of the adjacent panels is given in Fig. 46. It should be remembered that a resultant shear with the same sign as the dead-load shear causes the main diagonal to act, while a resultant shear of opposite sign to that of the dead-load shear causes the counter to act. The members



BRIDGE OVER MISSISSIPPI RIVER AT THEBES, ILLINOIS

Noble & Modjeski, Engineers, Chicago, Ill. Built in 1904-05. Consisting of two fixed spans, each 521 ft. 2 in.; four cantilever arms, each 152 ft. 6 in.; and three suspended spans, each 366 ft. For views of looking through and approach, see pages 10 and 75.

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acting are shown, and a section 4 – 4 can be passed. The dead-load shear at this section is 105 – 3 × 20 – 4 × 10 = +5.0; and accordingly,

$$-U_4L_4 + 5.0 = 0.$$

Therefore,

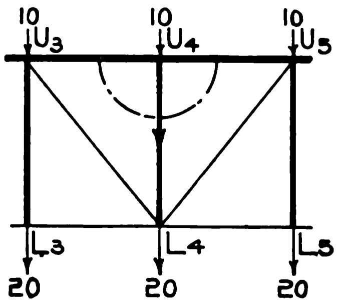
$$U_4L_4 = +5.0 = \text{Dead-load stress in this case.}$$

The live-load stress which acts at the same time is:

$$-U_4L_4 - 15.6 = 0 \qquad \therefore U_4L_4 = -15.6,$$

the term –15.6 representing the live-load shear on the section 4 – 4. This is not the minimum stress, as will next be shown, but it illustrates the fact that the loading for minimum live-load shears does not always give the minimum live-load stress.

By loading L_1, the live-load shear in the second panel, and likewise all others from this to the right support, will be –7.8. The total shears, together with their sign, and also the members they cause to act, are given in Fig. 47. The minimum live-load stress in U_4L_4 is found to be zero, and the dead-load stress is – 10, as is derived by passing a circular section around U_4, the equation being as follows:



d.l.v.	+ 15.0	– 15.0
l.l.v.	– 7.8	– 7.8
Total v.	+	–

Fig. 47. Stress Diagram for Vertical in Span of Fig. 42.

$$-\text{Live load at } U_4 - U_4L_4 = 0 \qquad \therefore U_4L_4 = 0 \text{ for live load.}$$
$$-\text{Dead load at } U_4 - U_4L_4 = 0 \qquad \therefore U_4L_4 = -10.0 \text{ for dead load.}$$

A diagram of half the truss should now be made, and all dead and live load stresses placed upon it, and these should be combined so as to form the maximum and the minimum stresses. Such a diagram, together with all stresses, is given in Fig. 48.

The stresses are written in the following order: Dead load, maximum live load, minimum live load, the maximum, and the minimum. In the chord and end-post stresses, there is no minimum live-load stress recorded, it being zero. Where pairs of stresses occur simultaneously, a bent arrow connects them.

40. The Howe Truss. The physical make-up of the Howe truss

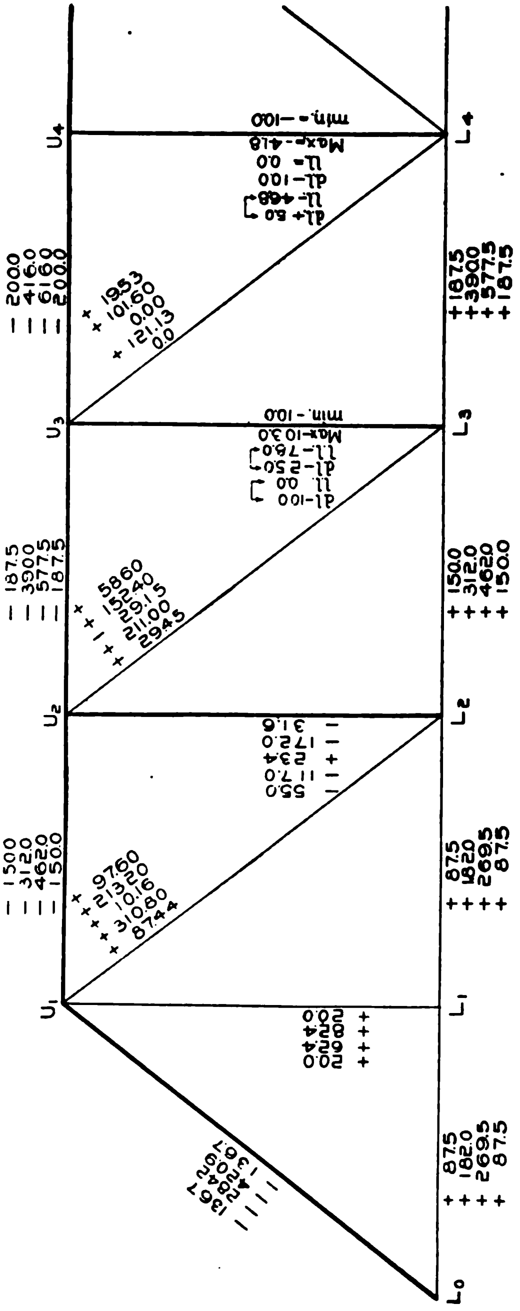


Fig. 48. Stress Diagram of Half-Span of Fig. 42, Pratt Truss, Single Track.

differs from that of the Pratt in that the diagonals are made to stand compression only, and the verticals can stand tension only. In the Pratt truss it was found that none of the intermediate posts could be brought into tension by any loading. In the Howe truss it will be found that none of the verticals can be brought into compression.

Let it be required to determine the stresses in a Howe truss of the same span, height, and loading as the Pratt truss of Article 39. An outline diagram is given in Fig. 49. The dead-load shears and the maximum and minimum live-load shears will be the same as for the Pratt truss, and they are:

Dead-Load V	+ Live-Load V	– Live-Load V
V_1 + 105.0	+ 218.4	– 0
V_2 + 75.0	+ 163.8	– 7.8
V_3 + 45.0	+ 117.0	– 23.4
V_4 + 15.0	+ 78.0	– 46.8
V_5 – 15.0	+ 46.8	– 78.0

Inspection of these shows that counters are required in the 4th and 5th panels (see Article 37).

The dead-load lower chord stresses will be computed by the tangent method (see Article 31), the section being $y - y_1$, etc. The tangent of ϕ is $25 \div 30 = 0.8333$. The stresses may be conveniently tabulated as follows:

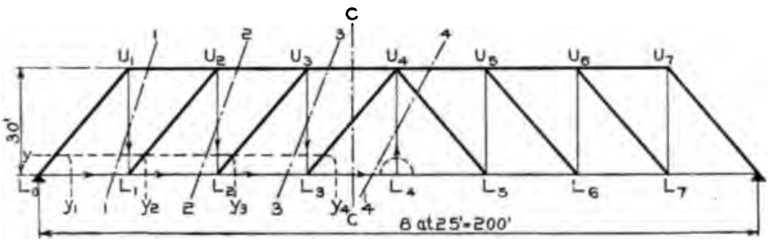


Fig. 49. Outline Diagram of 8-Panel Single-Track Howe Truss Span.

Dead-Load Chord Stresses (Lower Chord)

Member	Section	Stress Equation	Stress
L_0L_1	$y - y_1$	$-105.0 \times 0.8333 + L_0L_1 = 0$	+ 87.5
L_1L_2	$y - y_2$	$-(105.0 + 75.0)\,0.8333 + L_1L_2 = 0$	+ 150.0
L_2L_3	$y - y_3$	$-(105.0 + 75.0 + 45)\,0.8333 + L_2L_3 = 0$	+ 187.5
L_3L_4	$y - y_4$	$-(105.0 + 75.0 + 45.0 + 15.0)\,0.8333 + L_3L_4 = 0$	+ 200.0

A simple method for the determination of the upper chord stresses, is to pass a section and to equate the sum of the horizontal forces. Pass section 1 – 1. The only horizontal forces are the stresses in L_0L_1 and U_1U_2; and as these are parallel, one must be equal and opposite to the other. In a like manner the stresses in the other sections of the top chord are found. The stresses are:

$$U_1U_2 = -L_0L_1 = -(+\ 87.5) = -\ 87.5$$
$$U_2U_3 = -L_1L_2 = -(+150.0) = -150.0$$
$$U_3U_4 = -L_2L_3 = -(+187.5) = -187.5$$

A consideration of the Pratt truss shows that this method can be applied to it in determining the chord stresses.

As it is known that the diagonal web members are in compression under the dead load which produces a positive shear in the left half of the truss, it is evident that positive live-load shears will produce compressive stresses, and negative live-load shears tensile stresses, in the diagonals in the left half of the truss. Also, from Article 30, the stress in a diagonal is $V \sec \phi$. The stresses can now be written directly without the aid of the stress equation:

$$L_0U_1 = -105.0 \times 1.302 = -136.70$$
$$L_1U_2 = -\ \ 75.0 \times 1.302 = -\ \ 97.60$$
$$L_2U_3 = -\ \ 45.0 \times 1.302 = -\ \ 58.60$$
$$L_3U_4 = -\ \ 15.0 \times 1.302 = -\ \ 19.53$$

Likewise the stresses in the verticals can be written directly, remembering that here the secant is unity, and that the shear at the section cutting the member is to be used, not forgetting that $\frac{1}{3}$ of the dead panel load is applied at the top panel points. The shears and stresses are:

$$V_{1-1} = +105.0 - 10 = +95.0 \qquad U_1L_1 = +95.0$$
$$V_{2-2} = +105.0 - 20 - 2 \times 10 = +65.0 \qquad U_2L_2 = +65.0$$
$$V_{3-3} = +105.0 - 2 \times 20 - 3 \times 10 = +35.0 \qquad U_3L_3 = +35.0$$

The member U_4L_4 cannot be easily determined by passing a section 4 – 4, for this cuts four members. It is determined by passing a circular section about the point L_4, the equation being $+U_4L_4 - 20.0 = 0$, from which $U_4L_4 = +20.0$, which is equal to the dead panel load at the point L_4.

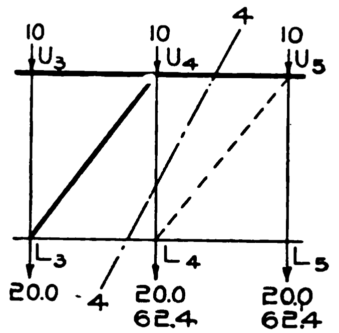
The live-load chord stresses are determined by multiplying the dead-load chord stresses by the ratio of the live to the dead loads. This has been found to be equal to 12.08. The live-load chord stresses are found to be:

$$L_0L_1 = +182.0 \qquad U_1U_2 = -182.0$$
$$L_1L_2 = +312.0 \qquad U_2U_3 = -312.0$$
$$L_2L_3 = +390.0 \qquad U_3U_4 = -390.0$$
$$L_3L_4 = +416.0$$

As the character of the stresses which can be taken by the diagonals and the verticals is known, the maximum and minimum live-load stresses can be written without first writing the stress equations. The maximum live-load stresses are:

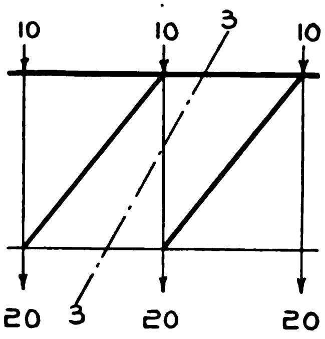
$$
\begin{array}{llll}
L_0U_1 = -218.4 \times 1.302 = -284.36 & U_1L_1 = +218.4 \\
L_1U_2 = -163.8 \times 1.302 = -213.27 & U_2L_2 = +163.8 \\
L_2U_3 = -117.0 \times 1.302 = -152.33 & U_3L_3 = +117.0 \\
L_3U_4 = -\ 78.0 \times 1.302 = -101.56 & U_4L_4 = +\ 78.0
\end{array}
$$

It should be noted that when L_4 and all panel points to the right are loaded, the shears and the members acting are as shown in Fig. 50. The dead-load shear on the section $4-4$ is $+15.0$, less the load at U_4, or $+15.0 - 10.0 = +5.0$; and the equation of stress is $-U_4L_4 + 5.0 = 0$, from which $U_4L_4 = +5.0$. Thus it is seen that in this case the dead-load stress is $+5.0$ when the live-load stress is $+78.0$.



d.l.v.	+ 15.0	− 15.0
l.l.v.	− 7.8	+ 15.6
Total v.	+	+

Fig. 50.



d.l.v.	+ 45.0	+ 15.0
l.l.v.	− 7.8	− 7.8
Total v.	+	+

Fig. 51.

Stress Diagrams for Members of Howe Truss Span of Fig. 49.

The maximum stresses in the counters (see Article 37) are:

$$(-46.8 + 15.0)\ 1.302 = -41.4.$$

The minimum live-load stresses are now written as follows:

$$
\begin{array}{ll}
L_1U_2 = +\ 7.8 \times 1.302 = +10.16 & U_1L_1 = 0 \\
L_2U_3 = +23.4 \times 1.302 = +29.15 & U_2L_2 = -7.8 \\
L_3U_4 = 0 & \left.\begin{array}{l} U_3L_3 \\ U_4L_4 \end{array}\right\} \text{See discussion following.} \\
U_3L_4 = 0 &
\end{array}
$$

If live panel loads were placed at points L_1, L_2, and L_3 the live-load shear in $c-c$ would be -46.8; and the dead-load shear being $+15$, the counter would act, and the stress in U_3L_3 would be tensile and equal to the sum of the dead and live panel loads which are at its lower end L_3. If points L_1 and L_2 had live panel loads on them, the resultant shear in $c - c$ would be $-23.4 + 15.0 = -8.4$; the counter would act, and the stress in U_4L_4 would be tensile and equal to the dead panel load which is at L_3. There being no live panel load at L_3, the live-load stress in U_3L_3 would be zero under this loading. If a live panel load be placed at L_1 only, then the shears and the members acting will be as shown in Fig. 51, and V_{3-3} for dead load $= +45.0 -$ the load at U_3, or $= 45 - 10 = +35.0$. The V_{3-3} for live load $= -7.8$, and the stress equation $- U_3L_3 - 7.8 = 0$, from which $U_3L_3 = -7.8$. So this live-load compression stress of 7 800 pounds occurs at the same time as the dead-load tensile stress of 45 000 pounds.

By loading various groups of panel points in succession and determining the resulting live-load stresses in U_4L_4, it will be found that under no loading can a negative live-load stress be produced. The minimum live-load stress is therefore zero, and occurs when there is no live load on the bridge.

The stresses should now be placed on an outline diagram similar to that of Fig. 48, and the stresses in corresponding members compared with those in that figure. This is left for the student.

41. **Bowstring and Parabolic Trusses.** A bowstring truss is shown in Fig. 13, the full lines representing the main members, which are the members under stress by the dead load. The dotted members represent counters which may be stressed by the action of the live load.

As before mentioned, the stresses in the chords and also in the webbing are quite uniform. When the end supports and the panel points lie on the arc of a certain curve, called a parabola, then, under full load, the stresses in all panels of the lower chord are equal; the stress in all verticals is tensile and is equal to the panel load at the lower end; and the stress in all diagonals is zero. Under partial load, the stresses in the webbing are exceedingly small, and the chord stresses remain almost equal.

If it is desired to have a parabolic truss, first decide upon the length of span, the number of panels, and the height at the center. The height of any vertical post is given by the formula:

$$h = H - \frac{4Hd^2}{l^2},$$

in which,

H = Approximate height at center;
d = Distance of vertical post from center;
l = Span;
h = Height of vertical post sought.

All distances are in feet. Suppose, as an example, that it was desired to determine the heights of the vertical posts in an 8-panel parabolic truss of a height approximately equal to 24 feet. One-half the truss is shown in Fig. 52. At the center, $d = 0$, and the equation reduces to $h = H$, which is 24 feet. For U_3L_3, $d = 20$; then,

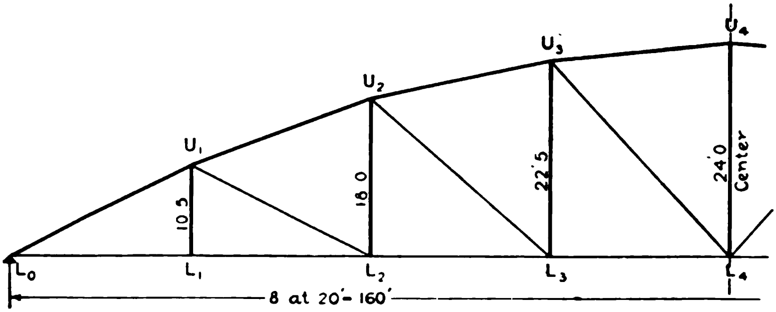


Fig. 52. One-Half of 8-Panel Parabolic Truss.

$$h = 24 - \frac{4 \times 24 \times 20^2}{160^2},$$

from which,

$$h = 22.5 \text{ feet.}$$

For U_2L_2,

$$d = 40$$

$$h = 24 - \frac{4 \times 24 \times 40^2}{160^2} \qquad h = 18.0 \text{ feet.}$$

For U_1L_1,

$$d = 60$$

$$h = 24 - \frac{4 \times 24 \times 60^2}{160^2} \qquad h = 10.5 \text{ feet.}$$

Inspection of the above results shows that the span or the center height must become quite great before the clearance at U_1L_1 will be sufficient to allow the traffic to pass under a portal bracing at this point. For this reason these trusses are usually built as through trusses with bracing on the outside of the truss, which connects to the floor-beams extended.

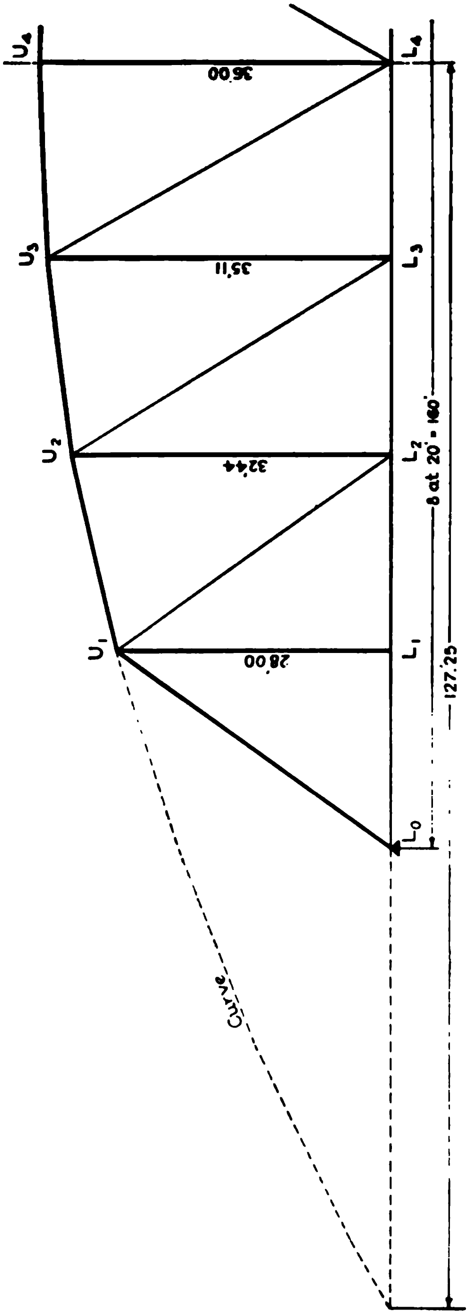


Fig. 53. Calculation of Heights of Vertical Posts in Parabolic Truss.

In the bowstring truss, the panel points of the top chord usually lie on the arc of a parabola which does not pass through the supports. For example, suppose that it was decided to have the span and panels the same as shown in Fig. 52, but the height at L_1 was to be 28 feet, and at L_4 36 feet. By substituting these values in the equation just given, and solving for l, the place will be determined where the parabolic curve cuts the lower chord extended, and the lengths of the vertical posts may be computed as before. Substituting these results:

$$28 = 36 - \frac{4 \times 36 \times \overline{60}^2}{l^2}$$

$$(-36 + 28)\,l^2 = -4 \times 36 \times \overline{60}^2$$

$$l = \sqrt{\frac{4 \times 36 \times \overline{60}^2}{8}} = 254.5,$$

which shows that the arc cuts the lower chord extended at a point 254.5 ÷ 2 = 127.25 feet from the center of the span (see Fig. 53).

The other vertical posts are:

$U_3L_3 \qquad h = 36 - \frac{4 \times 36 \times 20^2}{254.5^2} = 35.11$ feet;

$U_2L_2 \qquad h = 36 - \frac{4 \times 36 \times 40^2}{254.5^2} = 32.44$ feet;

$U_1L_1 \qquad h = 36 - \frac{4 \times 36 \times 60}{254.5^2} = 28.00$ feet, which checks.

The analysis of a bowstring truss will now be given. Both the maximum and minimum stresses will be determined, as reversal of stresses is liable to occur in the intermediate posts. The loading for minimum live-load stresses can be ascertained only by trial, care being taken to compute the dead-load stresses for the arrangement of web members caused by that particular live loading.

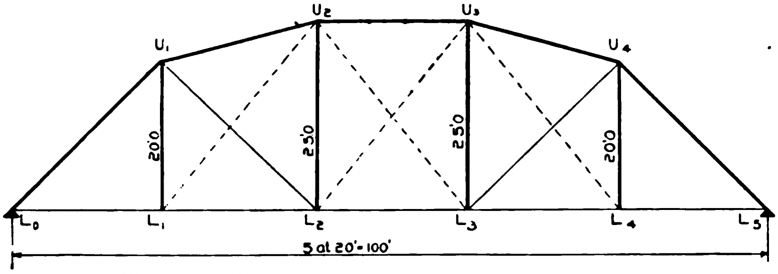


Fig. 54. Outline Diagram of 5-Panel Bowstring Truss Span.

Let it be required to determine the maximum stresses in the 5-panel 100-foot bowstring truss shown in Fig. 54, remembering that the diagonals take only tension. The height of U_1L_1 is 20 feet, and of U_2L_2 25 feet. The dead panel load is 17 200 pounds, and the live panel load is 50 000 pounds. The full lines show the main members which act under dead-load stress, and the dotted lines show the counters which may act under the action of the live load. One-third of the dead panel load, or 5 730 pounds, is taken as acting at the upper panel points, while the remainder, 11 470 pounds, acts at the lower

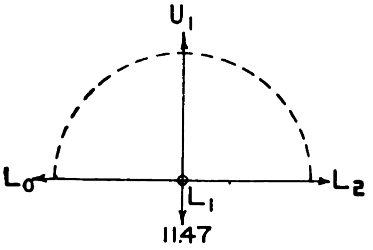


Fig. 55. Resolution of Forces around Panel Point in Bowstring Truss of Fig. 54.

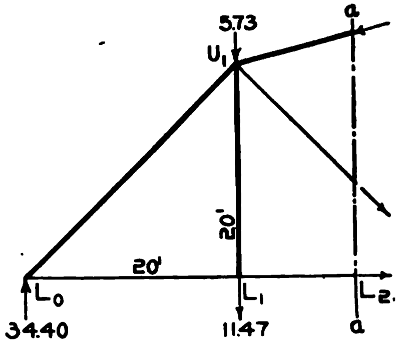


Fig. 53.

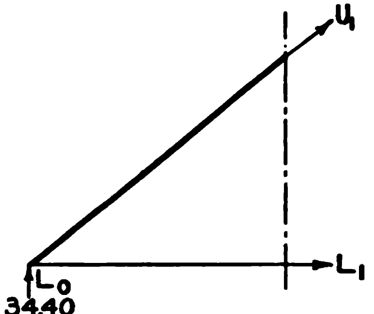


Fig. 58.

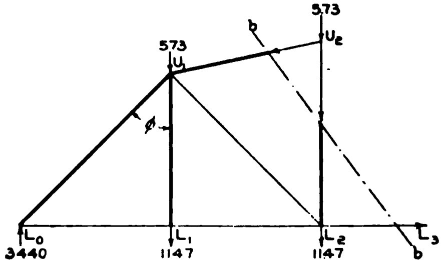


Fig. 57.

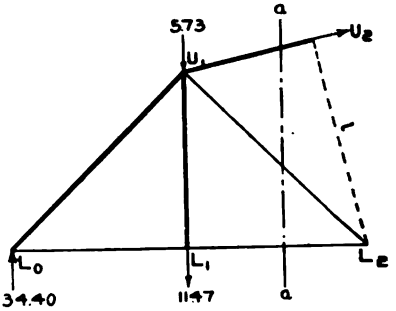


Fig. 59.

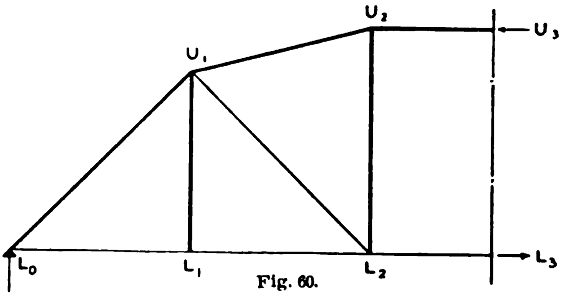


Fig. 60.

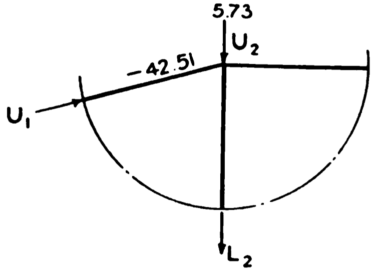


Fig. 61.

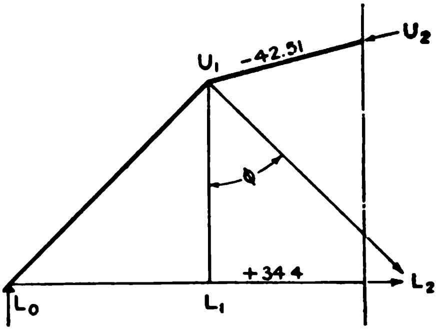


Fig. 62.

Analysis of Stresses in Various Members of the Bowstring Truss of Fig. 54.

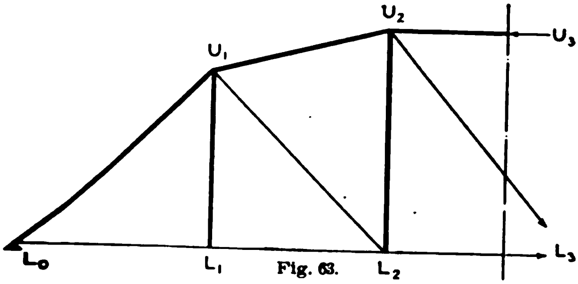


Fig. 63.

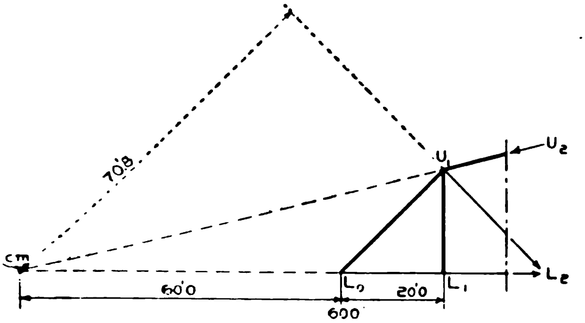


Fig. 64.

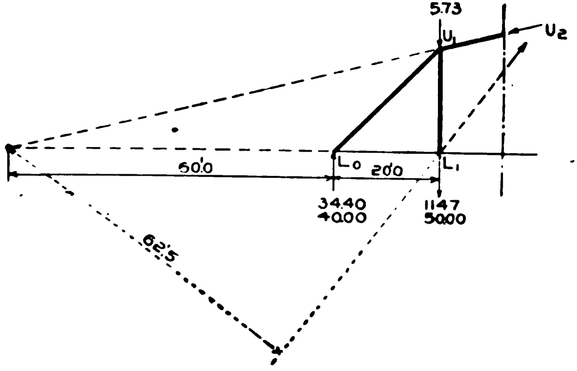


Fig. 65.

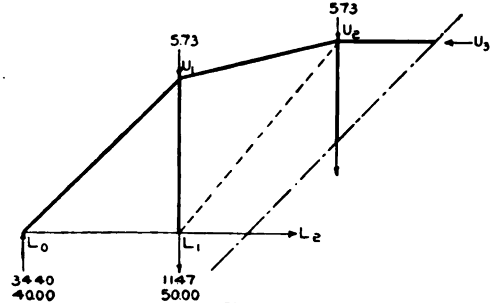


Fig. 66.

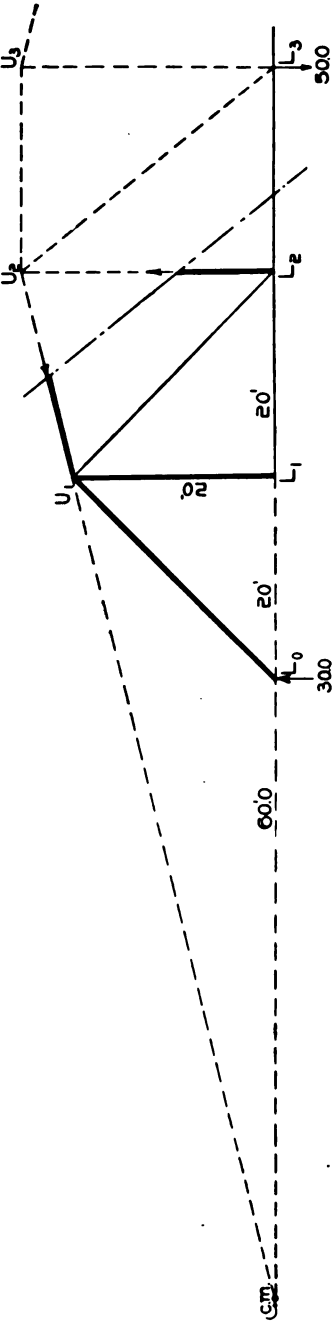


Fig. 67.

Analysis of Stresses in Various Members of the Bowstring Truss of Fig. 54.

Articles 27, 28, and 29 should be carefully reviewed before going further. The shear times the secant method cannot be conveniently employed for the live-load stresses in the members U_1L_2 and L_1U_2, as the section will cut the member U_1U_2, and the vertical component of its stress must be reckoned with in the stress equation. The method of moments as illustrated in Fig. 27, Article 29, will be used for these members.

The dead-load reaction is $2 \times 17.2 = +34.4$. The dead-load chord stresses should first be computed.

By resolving the horizontal forces around L_1, it is seen that $L_0L_1 = L_1L_2$ (see Fig. 55). Passing the section $a - a$, taking the center of moments, at U_1, and stating the equation of the moments to the left of the section, there results (see Fig. 56):

$$+34.4 \times 20 - L_1L_2 \times 20 = 0 \qquad \therefore L_1L_2 = +34.4$$

For L_2L_3, the section $b - b$ is passed; the center of moments is at U_2; and the equation of the moments to the left of this section (see Fig. 57) is:

$$+34.4 \times 2 \times 20 - (11.47 + 5.73)\,20 - L_2L_3 \times 25 = 0 \quad \therefore L_2L_3 = +41.26.$$

By passing a vertical section cutting L_0L_1 and L_0U_1, the stress in L_0U_1 can be determined by taking the sum of the vertical forces to the left and equating them to the vertical component of the stress (see Fig. 58). The equation is:

$$+34.4 + L_0U_1 \times 0.707 = 0, \text{ from which } L_0U_1 = -34.4 \times 1.414 = -48.7.$$

A section $a - a$ (Fig. 59) shows that the center of moments for U_1U_2 is at U_2; and stating the moments of the stress, and the forces to the left of the section, there results an equation in which an unknown lever arm enters. This lever arm l is readily computed to be 24.28 feet, and the equation can now be written:

$$+34.4 \times 2 \times 20 - (11.47 + 5.73)\,20 + U_1U_2 \times 24.28 = 0$$
$$\therefore U_1U_2 = -42.51.$$

The stress in U_2U_3 is determined by passing a vertical section in the 3d panel, and taking the sum of the horizontal forces. As there is no dead-load stress in the members L_2U_3 and U_2L_3, their components will be zero. Therefore (see Fig. 60) it is evident that U_2U_3 must be equal and opposite to L_2L_3 and will be equal to -41.26.

By reference to Fig. 55, the stress in L_1U_1 is seen to be tensile and equal to $+11.47$.

Pass a circular section around U_2 and take the sum of the vertical components, assuming that the stress in U_2L_2 acts away from the section. The length of U_1U_2 is $\sqrt{5^2 + 20^2} = 20.6$, and therefore the vertical component of U_1U_2 will be $(42.51 \div 20.6) \times 5 = 10.32$, which acts upward. The stress equation of U_2L_2 (see Fig. 61) is:

$$+10.32 - 5.73 - U_2L_2 = 0 \qquad \therefore U_2L_2 = +4.59,$$

showing that a tensile stress occurs in U_2L_2 when all panel points are loaded.

The simplest method of ascertaining the stress in U_1L_2 is to pass a vertical section cutting members as shown in Fig. 62, and to equate the horizontal forces and stresses. The horizontal component of U_1U_2 is:

$$\frac{42.51}{20.6} \times 20 = 41.30, \text{ which acts toward the left.}$$

The equation of stress is, then:

$$\begin{aligned} -41.30 + 34.40 + U_1L_2 \sin \phi &= 0; \text{ but } \sin \phi = 0.707; \\ U_1L_2 &= +6.90 \times 1.414 \\ &= +9.76 \end{aligned}$$

All the dead-load stresses being computed, the next operation will be to determine the live-load chord stresses. These are proportional to the dead-load stresses in the same ratio as the live panel load is to the dead panel load. This ratio is $50 \div 17.2 = 2.907$, and the chord and end-post live-load stresses are:

$$\begin{aligned} L_0U_1 &= -48.71 \times 2.907 = -141.7 \\ U_1U_2 &= -42.51 \times 2.907 = -123.6 \\ U_2U_3 &= -42.26 \times 2.907 = -123.0 \\ L_0L_2 &= +34.40 \times 2.907 = +100.2 \\ L_2L_3 &= +41.26 \times 2.907 = +120.3 \end{aligned}$$

Also, the stress in U_2L_2 when the live load covers the entire bridge is *not* 2.907×4.59, as it must be remembered that part of the dead load is at the panel points of the upper chord. Taking a circular section around U_2 (see Fig. 61), and noting that there is no load at U_2, it is seen that the stress in U_2L_2 due to live load is simply equal to the vertical component of the live-load stress of U_1U_2 and will be tensile. It is:

$$U_1U_2 = (123.6 \div 20.6) \times 5 = +30.0.$$

The maximum live-load stress in U_1L_1 is tensile, and equal to the live panel load at L_1 (see Fig. 55).

To obtain the maximum stress in U_2L_3, load L_3 and L_4. The shear V_3 will then be $\frac{50}{5}(1 + 2) = +30.0$. The section will cut the members as shown in Fig. 63, and the equation of stress will be:

$$+30.0 - U_2L_3 \cos\phi = 0; \text{ but } \cos\phi = \frac{25}{\sqrt{20^2 + 25^2}} = 0.782;$$

$$\therefore U_2L_3 = +38.4.$$

If panel points L_1 and L_2 were loaded, it is evident that the stress in L_2U_3 would be $+38.4$.

To obtain the maximum live-load stress in U_1L_2, a section is passed cutting U_1U_2, L_1L_2, and U_1L_2 (Fig. 64). The center of moments will be at the intersection of U_1U_2 and L_1L_2, and this point lies some place to the left of the support L_0. The lever arm of U_1L_2 will be the perpendicular distance from this point to the line U_1L_2 extended. The panel points L_2, L_3, and L_4 are loaded. The left reaction is then $(1 + 2 + 3)\frac{50}{5} = +60.0$. The lever arms are easily computed, and these, together with the members cut, are shown in Fig. 64. The equation of stress is:

$$-60.0 \times 60.0 + U_1L_2 \times 70.8 = 0 \qquad \therefore U_1L_2 = +50.80.$$

If a load were put on L_1 only, then the reaction at L_0 would be $\frac{4}{5} \times 50 = 40$; and the equation of stress would then be:

$$-40.0 \times 60.0 + 50 \times (60.0 + 20.0) + U_1L_2 \times 70.8 = 0 \quad \therefore U_1L_2 = -22.6.$$

As this is compression and greater than the dead-load stress, $+9.76$, a counter is required in that panel. In order to get the stress in the counter, it must be inserted, U_1L_2 being left out, and the dead and live load stresses computed and their difference taken. Fig. 65 gives the lever arms, center of moments, and the forces acting in this case.

The dead-load stress is:

$$-34.4 \times 60.0 + (11.47 + 5.73)(60 + 20) - L_1U_2 \times 62.5 = 0$$
$$\therefore L_1U_2 = -11.02;$$

and the live-load stress is:

$$-40 \times 60 + 50(60 + 20) - L_1U_2 \times 62.5 = 0 \qquad \therefore L_1U_2 = +25.60,$$

and the stress in the counter is the algebraic sum of these two, or $-11.02 + 25.60 = +14.58$.

When a live panel load is at L_1, L_1U_2 is acting, as has just been proved. As this load at L_1 causes a negative shear in all panels to the right, this negative shear in the center panel will cause L_2U_3 to act. A section may now be passed as shown in Fig. 66, and the stress equations for U_2L_2 written:

For dead load, $+34.4 - 11.47 - 2 \times 5.73 - U_2L_2 = 0 \quad \therefore U_2L_2 = +11.47$
For live load, $+40.0 - 50.0 - U_2L_2 = 0 \quad \therefore U_2L_2 = -10.00$
Total $= +\ 1.47$

This is evidently not a maximum for U_2L_2, for when a full live load was on the span, the stress was +30.0 due to live load and +4.59 due to dead load.

It might be well to consider what effect is produced by loading L_3 and L_4. The loading of L_2 and L_1 need not be considered, since it is evident that, as this causes the total shear in panel 2 to be positive and the total shear in panel 3 to be negative, therefore U_1L_2 and L_2U_3 will act, and this causes a tensile stress in U_2L_2 equal to the vertical components of the dead and live load stresses in U_1U_2 less the dead panel load at U_2. With a live panel load at L_3 and L_4, the left reaction is $\frac{50}{5}(1 + 2) = +30.0$. The section, the live-load forces, the center of moments, and the members acting are shown in Fig. 67. The dead-load stress in U_2L_2 will be the same as when the truss has no live load on it. The stress equation for the live load is:

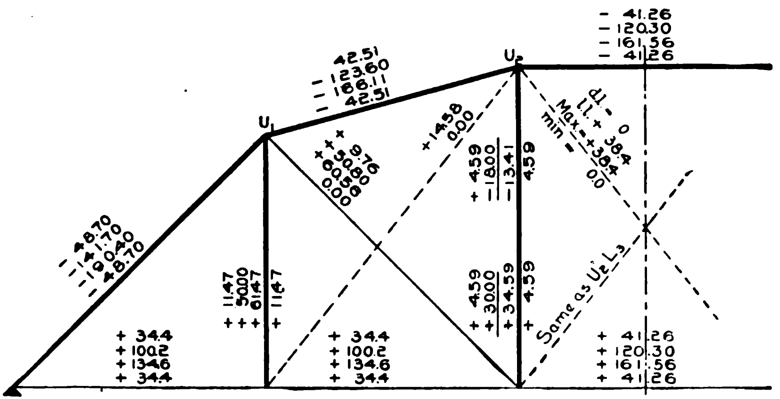


Fig. 68. Stress Diagram of Half-Span of Parabolic Truss of Fig. 54.

$$-60 \times 30 - (60 + 20 + 20) \times L_2U_2 = 0 \qquad \therefore L_2U_2 = -18.0.$$

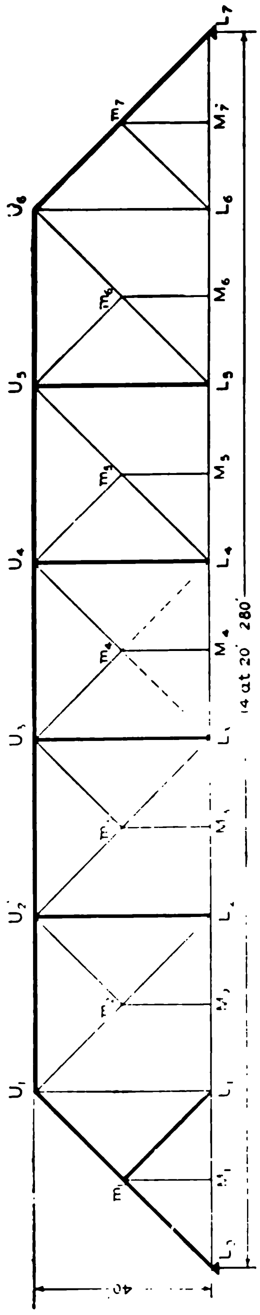


Fig. 69. Outline Diagram of 14-Panel Baltimore Truss Span.

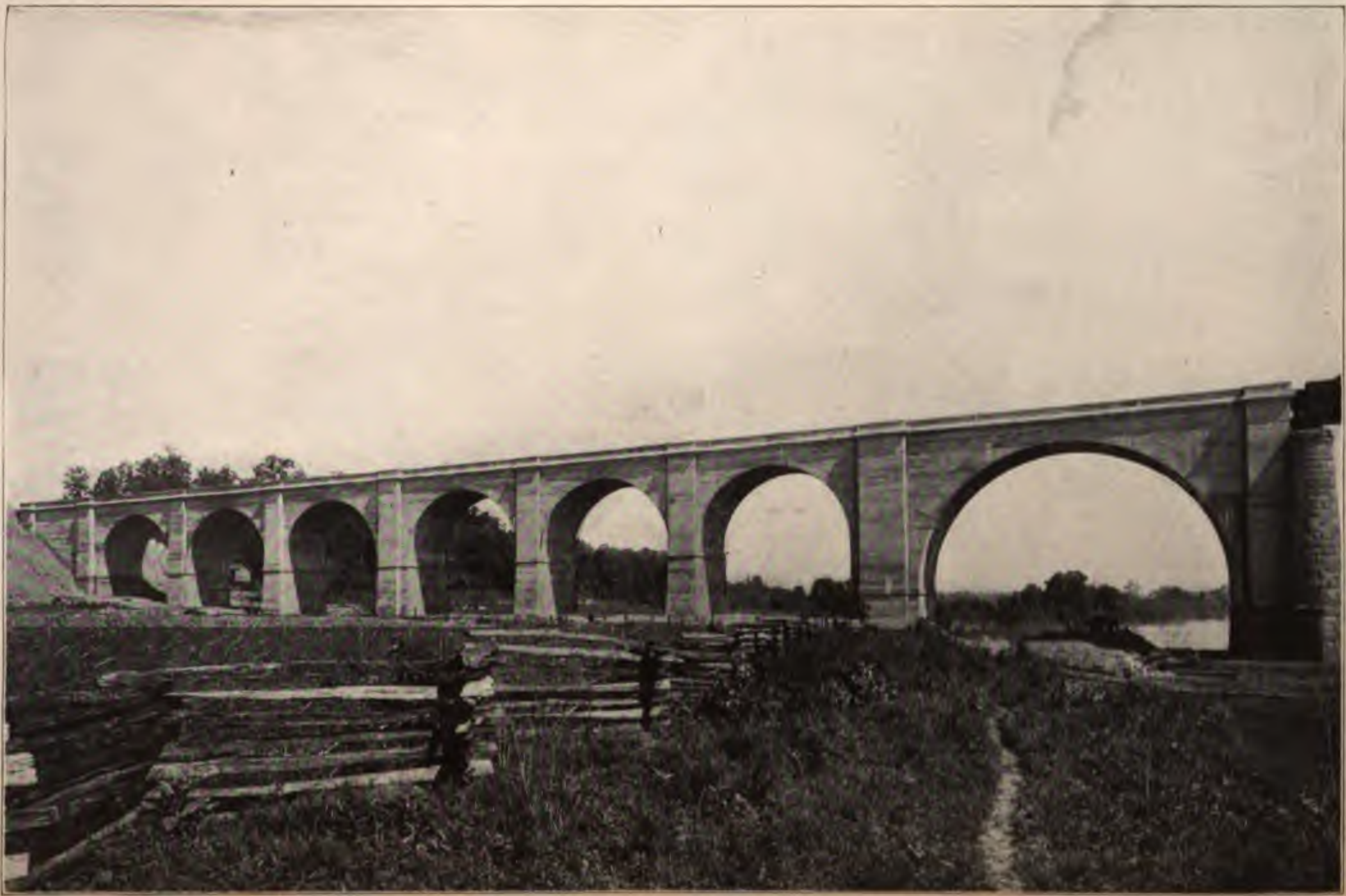
The dead-load stress being +4.59, this stress of −18.0 causes a reversal of stress in the vertical. For this reason the verticals of bowstring trusses are, like web members of Warren trusses, built so as to take either tension or compression. The minimum stresses in the diagonals will be zero, for when one diagonal in a panel is acting, the other is not.

The diagram of half of the truss in Fig. 68 gives all the stresses.

It is to be noted by the student, that in some cases one method for the determination of stresses is preferable to others in that it saves labor of computation. The analysis of the truss of Fig. 68 illustrates this fact.

42. The Baltimore Truss. Baltimore trusses are of two classes—those in which the half-diagonals, called *sub-diagonals*, are in compression, and those in which the sub-diagonals are in tension. The latter class is the one most usually built, as it is more economical on account of many of its members being in tension, in which case these members are cheaper and easier to build than if they were compression members. Fig. 14 shows both types of truss. The Baltimore truss does not have a simple system of webbing, and for that reason the analysis is here presented. As the tension sub-diagonal truss is the type in most common use, its analysis will be given.

Let it be required to compute the maximum stresses in the 14-panel 280-foot span of Fig. 69. The height is 40 feet, the dead panel load 24 000 pounds,



MISSOURI APPROACH OF BRIDGE OVER MISSISSIPPI RIVER AT THEBES, ILLINOIS
Noble & Modjeski, Engineers, Chicago, Ill. For other views of this bridge, see pages 10 and 58

and the live panel load 40 000 pounds. One-third of the dead panel load is applied at the upper ends of the long verticals and also of the half-verticals. These half-verticals are designated as *sub-verticals*. Attention is called to the system of notation used for the ends of the sub-verticals. The full lines in Fig. 69 represent the main members, being stressed by dead load only. The heavy lines indicate those members that take compression, the light lines those that take tension, and the broken lines the counter-braces. In this, as in nearly all Baltimore trusses, the diagonals make an angle of 45 degrees with the vertical.

The dead and the positive live-load shears in the various panels should be computed. They are:

DEAD-LOAD V		LIVE-LOAD V
V_1	+156.00	$V_1 = (1 + \ldots 13)\frac{40}{14} = +260.00$
V_2	+132.00	$V_2 = (1 + \ldots 12)\frac{40}{14} = +223.00$
V_3	+108.00	$V_3 = (1 + \ldots 11)\frac{40}{14} = +188.50$
V_4	+ 84.00	$V_4 = (1 + \ldots 10)\frac{40}{14} = +157.20$
V_5	+ 60.00	$V_5 = (1 + \ldots 9)\frac{40}{14} = +128.50$
V_6	+ 36.00	$V_6 = (1 + \ldots 8)\frac{40}{14} = +102.80$
V_7	+ 12.00	$V_7 = (1 + \ldots 7)\frac{40}{14} = +80.00$

It is only necessary to determine the negative live-load shear in panels 5 and 7, in order to ascertain if there is a counter required. These shears are:

$$-V_5 = (10 + 11 + 12 + 13)\frac{40}{14} - 4 \times 40 = -28.60$$

$$-V_7 = (8 + 9 + 10 + 11 + 12 + 13)\frac{40}{14} - 6 \times 40 = -60.00$$

From a comparison of these with the dead-load shears, it is seen (see Article 37) that a counter is required in panel 7 only.

The dead-load stresses are first to be computed. The stress in any sub-vertical is found by passing a circular section around its lower end, and equating the sum of the vertical forces, assuming in this, as in all cases, that the unknown stress acts away from the [illegible] Fig. [illegible] gives the motion, the [illegible]

[illegible]

[illegible] lower end, it [illegible] is the same, a [illegible]

[illegible] determined by [illegible] The com- [illegible] Fig. [illegible] The

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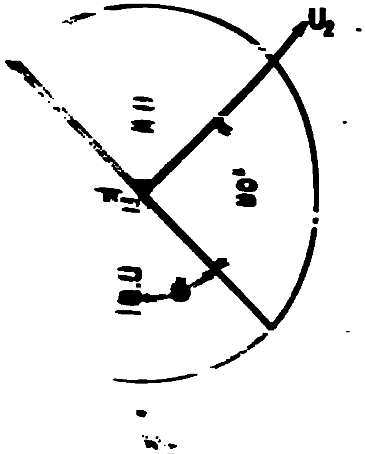


[illegible]

[illegible] and dividing [illegible]

[illegible] sub-diagonals is equal to [illegible] the angle ϕ. It also shows that the [illegible] of the sub-diagonal is equal to *one-half the panel load.* This fact should be remembered, as it will be frequently used further on.

In a similar manner, the stress in all the tension sub-diagonals will be found to be the same, +16.96, and the stress in the compression sub-diagonal m_1L_1 is -16.96.

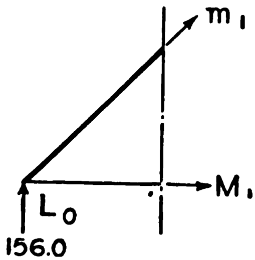


Fig. 72. Stress in Diagonal of Baltimore Truss.

The stress in the member L_0m_1 and in the upper half of any main diagonal (*i. e.*, U_1m_2, U_2m_3, and U_3m_4) is determined as in the diagonals of the Pratt or Howe truss, for the section passed cuts but one member, which has a vertical component. Take L_0m_1 (see Fig. 72). Then $+156.0 + L_0m_1 \cos 45° = 0$, from which $L_0m_1 = -220.5$. For U_1m_2 the section is passed as in Fig. 73, and the equation of stress is $+V_3 - U_1m_2 \cos 45° = 0$, or $+108.0 - U_1m_2 \times 0.707 = 0$, from which $U_1m_2 = +152.9$.

In a similar manner,

$$U_2m_3 = +60.0 \div 0.707 = +84.84;$$
$$U_3m_4 = +12.0 \div 0.707 = +16.96.$$

The stresses in m_1U_1, m_2L_2, and m_3L_3 may be determined by resolving the forces about m_1, m_2, and m_3; but a neater solution is to pass a vertical section cutting the member whose stress is desired, and to equate to zero the shear and the vertical components of all

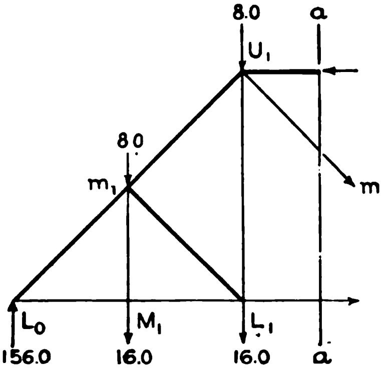


Fig. 73. Stress in Upper Half of Main Diagonal of Baltimore Truss.

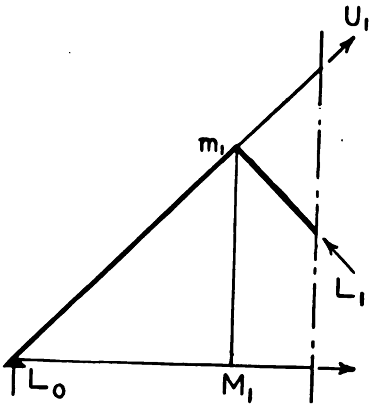


Fig. 74. Stress in Diagonal of Baltimore Truss.

[illegible] The section for m_1U_1 is [illegible]:

$$[illegible] = 0$$

[illegible] and therefore,

$$[illegible] = 0$$

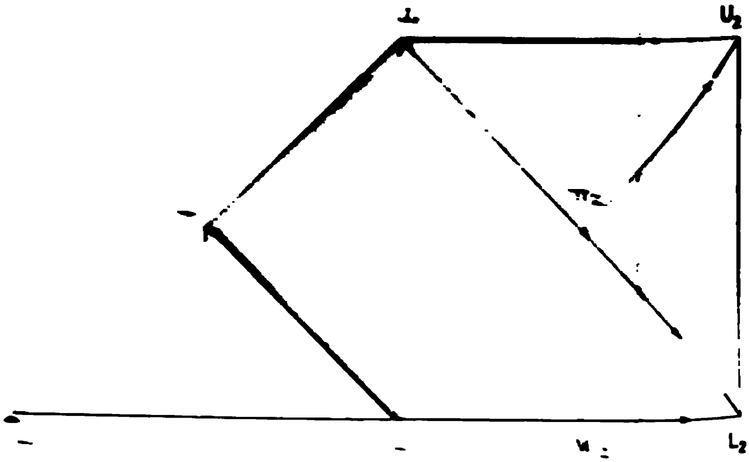
$$[illegible] = -[illegible]$$

[illegible] shown in Fig. [illegible] and the stress [illegible]

$$[illegible] - V_4 = 0$$

$$[illegible] = 0$$

$$[illegible] = -[illegible]$$



[illegible] Baltimore Truss

[illegible] m_3U_3, m_3L_3, [illegible]

[illegible]

[illegible] determined by resolving [illegible] Referring successively to [illegible] are:

$$[illegible] = -28.0$$

$$[illegible] = -[illegible]0.0$$

$$[illegible] = -32.0$$

[illegible] and [illegible] being the vertical components of m_2L_2 and m_3L_3 respectively.

The chord stresses are easiest computed by considering the resolution of horizontal forces at the panel points. As the diagonals make an angle of 45° with the vertical, their horizontal and vertical components are equal. For instance, the horizontal component of the members L_0m_1, U_1m_2, and U_2m_3 are equal to the shear in that panel, which is their vertical component. At point L_0 (see Fig. 77), there results:

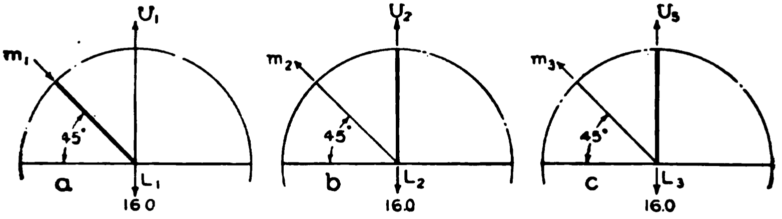


Fig. 76. Calculating Stresses in Verticals of Baltimore Truss of Fig. 69.

$$+L_0M_1 - \text{horizontal component of } M_1L_0 = 0; \text{ or,}$$
$$+L_0M_1 - 156 = 0$$
$$\therefore L_0M_1 = +156.0;$$

and from Fig. 70 it is evident that $L_0M_1 = M_1L_1$. At point L_1 (see Fig. 78), L_1M_2 is equal to M_1L_1, less the horizontal component of M_1L_1, and the equation is:

$$-156 + 12 + L_1M_2 = 0$$
$$\therefore L_1M_2 = +144.0; \text{ and } M_2L_2 = +144.0.$$

At point L_2 (see Fig. 79), L_2M_3 is equal to the sum of the horizontal components of M_2L_2 and m_2L_2; that is,

$$+L_2M_3 - 144.0 - 96.0 = 0$$
$$\therefore L_2M_3 = M_3L_3 = +240.0.$$

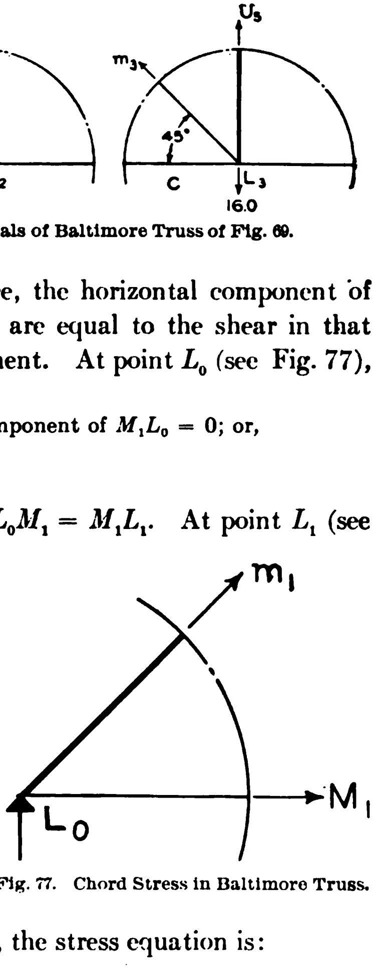


Fig. 77. Chord Stress in Baltimore Truss.

In a similar manner, at point L_3, the stress equation is:

$$+L_3M_4 - 240.0 - 48.0 = 0$$
$$\therefore L_3M_4 = M_4L_4 = +288.0.$$

At the upper panel point U_1 (see Fig. 80), there results the equation:

$$+U_1U_2 + \text{hor. comp. } U_1m_2 + \text{hor. comp. } m_1U_1 = 0;$$
$$U_1U_2 + 108.0 + 144 = 0; \text{ or, } U_1U_2 = -252.0.$$

For the member U_2U_3 (see Fig. 81), the equation is:

$$+U_1U_2 + U_2U_3 - \text{hor. comp. } m_2U_2 + \text{hor. comp. } U_2m_3 = 0$$
$$+252.0 + U_2U_3 - 12 + 60 = 0$$
$$\therefore U_2U_3 = -300.0.$$

In a similar manner, by resolving the horizontal forces at U_3, it will be seen that the action of m_3U_3 will neutralize that of U_3m_4, as

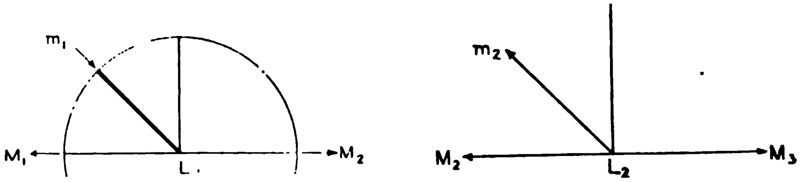


Fig. 78. Fig. 79.

Bottom Chord Stresses in Baltimore Truss.

they are equal and pull in opposite directions, and U_3U_4 is equal to $U_2U_3 = -300.0$.

The live-load stresses in the chords, the end-post, and the sub-diagonals are all proportional to the dead-load stresses in the same

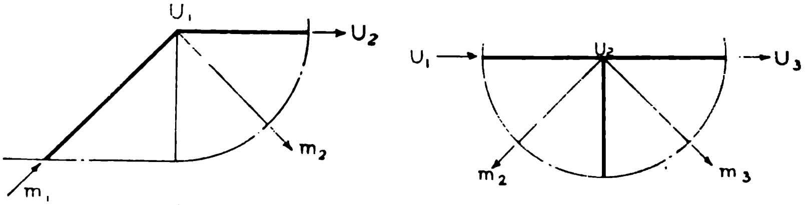


Fig. 80. Fig. 81.

Top Chord Stresses in Baltimore Truss.

ratio as the live panel load is to the dead panel load. This ratio is $\frac{40}{24} = 1.667$. By reference to Fig. 70, it will be seen that the live-load stress in the sub-verticals is $+40.0$ for each one. The following stresses can now be determined:

$$L_0m_1 = +220.5 \times 1.667 = +367.5$$
$$m_1U_1 = -203.6 \times 1.667 = -339.5$$
$$U_1U_2 = -252.0 \times 1.667 = -420.0$$
$$U_2U_3 = -300.0 \times 1.667 = -500.0$$
$$U_3U_4 = -300.0 \times 1.667 = -500.0$$

$$L_0L_1 = +156.0 \times 1.667 = +260.0$$
$$L_1L_2 = +144.0 \times 1.667 = +240.0$$
$$L_2L_3 = +240.0 \times 1.667 = +405.0$$
$$L_3L_4 = +288.0 \times 1.667 = +481.0$$

$$m_1L_1 = -\ 16.96 \times 1.667 = -\ 28.28$$
$$m_2U_2 = m_3U_3 = +16.96 \times 1.667 = +28.28$$

The vertical U_1L_1 will have its maximum live-load stress when points M_1 and L_1 are loaded, for these are the only loads which cause a stress in that member (see Fig. 76*a*). The equation is:

$$-\frac{40}{2} - 40 + U_1L_1 = 0,$$

from which,

$$U_1L_1 = +60.0.$$

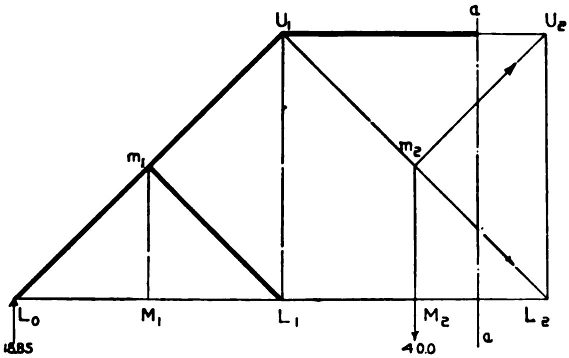


Fig. 82. Stress in Lower Half of Main Diagonal of Baltimore Truss.

The maximum live-load stresses in U_1m_2, U_2m_3, and U_3m_4 are obtained in a manner exactly like that used in obtaining dead-load stress, only the live-load positive shear is used. The stresses are:

$$-U_1m_2 \times 0.707 + 188.5 = 0 \qquad \therefore U_1m_2 = +266.5$$
$$-U_2m_3 \times 0.707 + 128.5 = 0 \qquad \therefore U_2m_3 = +181.5$$
$$-U_3m_4 \times 0.707 + 80.0 = 0 \qquad \therefore U_3m_4 = +113.1$$

In the determination of the maximum live-load stress in the lower halves of the main diagonals, m_2L_2, m_3L_3, and m_4L_4, one of the peculiarities of this truss becomes apparent. A section being passed as in Fig. 82, the panel point ahead of the section, and all between the section and the right support, must be loaded. This of course produces a stress in m_2U_2, and the vertical component of this enters the stress equation. The shear in the section $a - a$ *under this loading* is:

$$V_{a-a} = +188.5 - 40 = +148.5;$$

and the stress equation is:

$$-m_2L_2 \times 0.707 + \frac{40}{2} + 148.5 = 0$$

$$\therefore m_2L_2 = +238.0.$$

If the truss had been loaded from the section to the right, there being no load on M_2, no stress would result in m_2U_2, and the stress in m_2L_2

would have been $m_2L_2 = \dfrac{157.2}{0.707} = +222.2$. In a similar manner, by loading successively points M_3 and to the right, and M_4 and to the right, the stress equations of m_3L_3 and m_4L_4 are:

$$-m_3L_3 \times 0.707 + \frac{40}{2} + 128.5 - 40 = 0 \qquad \therefore m_3L_3 = +153.3$$

$$-m_4L_4 \times 0\ 707 + \frac{40}{2} + 80.0 - 40 = 0 \qquad \therefore m_4L_4 = +84.8$$

The maximum live-load stresses in the main verticals occur when the panel points to the right of the section which cuts the member under consideration are loaded. There being no load at the end of the sub-vertical just to the left of the section, there will be no stress in the sub-diagonal which the section cuts. The chords, of course, do not exert a vertical component; and so the only unknown term of the stress equation is the stress in the member itself. Fig. 83 shows how the section should be passed when U_2L_2 is considered. The stress equation is:

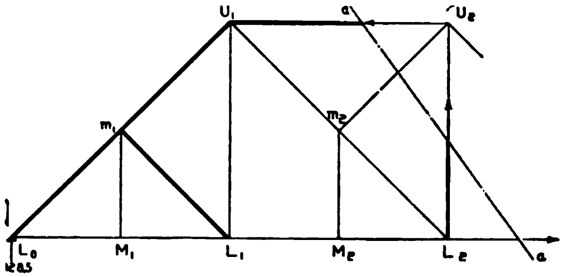


Fig. 83. Stress in Main Vertical of Baltimore Truss.

$$+U_2L_2 + V_{a-a} = 0; \qquad +U_2L_2 + 128.5 = 0; \qquad \therefore U_2L_2 = -128.5.$$

In a similar manner, by passing a section cutting U_2U_3, m_3U_3, U_3L_3, L_3m_4, and loading M_4 and to the right, it is seen that the stress equation for U_3L_3 is:

$$+U_3L_3 + 80.0 = 0 \qquad \therefore U_3L_3 = -80.0$$

The components of m_3U_3 and L_3m_4 are zero, as can readily be proved by solving for them under this loading.

Fig. 84 gives all the stresses, and they are written in order of dead load, live load, and maximum.

43. **Other Trusses.** The analysis of the foregoing trusses will enable one to solve any of the trusses of modern times. For the solution of the Whipple (sometimes called the "double-intersection Pratt") and others which are not mentioned in this text, the student

is referred to the textbooks of F. E. Turneaure and Mansfield Merriman.

ENGINE LOADS

44. Use of Engine Loads. It was formerly the custom for railroads to specify that the engine to be used in computing the stresses in their bridges should be one of their own which was in actual use. The engines of different roads were usually different both in regard to the weight on the various wheels and in regard to the number and spacing of the wheels. Of late years, considerable progress has been made towards the adoption of a typical engine loading as standard. These typical engines (see Fig. 17, Article 25) vary only in regard to the weights on the wheels, the number and spacing of wheels being the same in all engines.

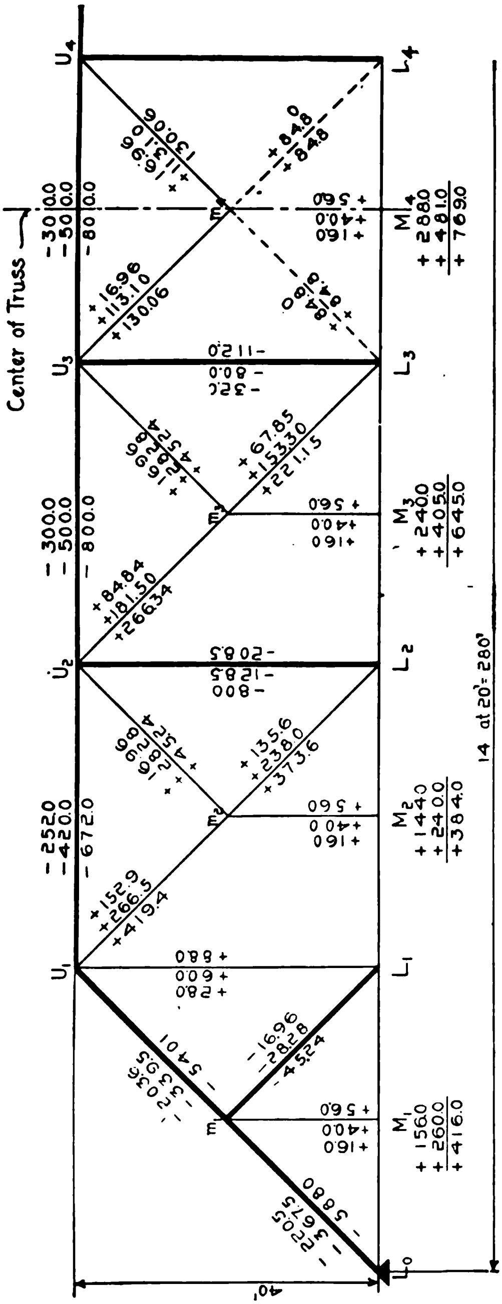


Fig. 84. Stress Diagram of Baltimore Truss of Fig. 69. Stresses indicated in order of Dead-Load, Live-Load, and Maximum.

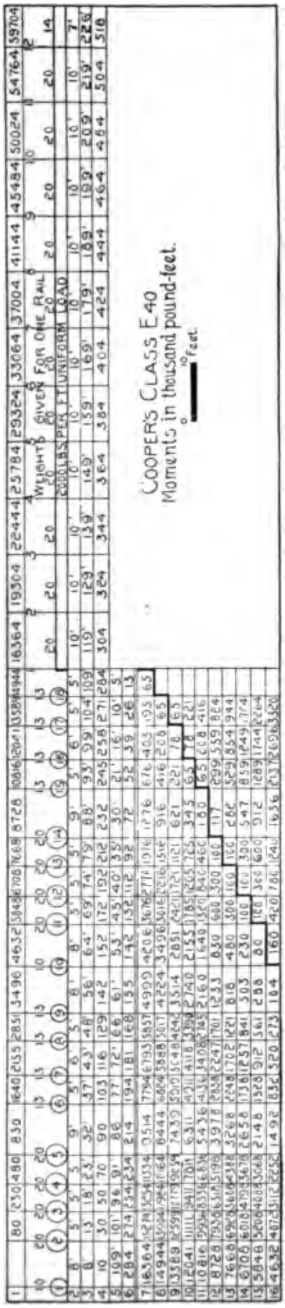

Weights given for one rail
2000 lbs per ft uniform load
Cooper's Class E40
Moments in thousand pound-feet.
0
10 Feet

Fig. 85. Diagram for Computing Stresses under Engine Loads.

The distance between wheels is an even number of feet, instead of an odd number of feet and inches and fractions thereof. For examples of loadings which are in almost universal use, consult the specifications of Cooper or Waddell.

The labor of computation of stresses when engine loads are used is considerably lessened by the use of the so-called *engine diagrams*. Fig. 85 gives a diagram which has been found very convenient. The first line at the top represents the bending moment of all the loads about the point to the right of it. All the loads are given in thousands of pounds, and all the moments are in thousands of pound-feet. The practice of writing results in thousands of pounds—or, in case of moments, in thousands of pound-feet or pound-inches—is to be recommended, as it saves the unnecessary labor of writing ciphers. Throughout this text this practice has been extensively followed, the stresses being written to the nearest ten pounds or one-hundred pounds as the case may be. For example, 6 433 may be written 6.43 or 6.4, the few pounds which are neglected making no appreciable difference in the design. The distances are in feet.

As an example of the use of the first line at the top, suppose that it is desired to find the moment of all the loads to the left of a certain point when wheel 6 (the numbers of the wheels are placed inside of the circles representing the wheels) is just over the point. The moment will be 1 640 000 pound-feet, which is obtained by reading off the 1 640 just to the right of the line through wheel 6.

When using the first line for values at sections in the uniform load, the values given represent the moment of all wheel and uniform loads about the points in the line or section to the left of the value given. For example, if it is desired to find the moment about a point in line 2, it will be 19 304 000 pound-feet, the value 19 304 appearing to the right of the line.

The line of figures below the wheels indicates the distances between any two wheels.

The third line of figures indicates the distance from the first wheel to the wheel to the right. For instance, 37 is the distance from wheel 1 to wheel 7.

The values in the fourth line indicate the sum total of all the loads to the left of the value given. For example, 245 signifies that the loads 1 to 15 inclusive weigh 245 000 pounds.

The values in lines 5 and 6 are similar to those of lines 3 and 4, except that the starting point is at the head of the uniform load. For example, 40 in line 5, and 112 in line 6, indicate that it is 40 feet from the head of the uniform load to the wheel 12, and that wheels 18 to 13 inclusive weigh 112 000 pounds.

The values in lines 7 to 16 indicate the value of the moment of all the wheels from the zigzag line up to and including the one to the left or the right, according as the value is to the left or the right of the zigzag line. For example, 2 745, line 11, indicates that the moments of wheels 8 to 14 inclusive about the zigzag line just under wheel 15, is 2 745 000 pound-feet; or the value 1 704, line 14, shows that the moments of wheels 13 to 18 about the zigzag line just under wheel 12 is 1 704 000 pound-feet.

When line 4 of figures is under the uniform load, the values refer to the vertical line to the right; thus 324 is the value of all loads to the left of line 3 about that line.

For values of moments at points which fall in between wheels, or at positions in the uniform load where the value of the moment is not given, a very important principle of applied mechanics is used. It is:

$$M_x = M' + Wx + \frac{wx^2}{2}.$$

in which,

M_a = Moment at section desired;
M' = Value of moment at preceding vertical line;
W = Sum total of all loads to the left of and at the point where M' is taken;
x = Distance from section under consideration to vertical line to which M' is referred;
w = Uniform load on the distance x.

Let it be desired, for example, to determine the moment at a point c, 3 feet to the right of wheel 13. The position of the loads is given in Fig. 86. The moment is:

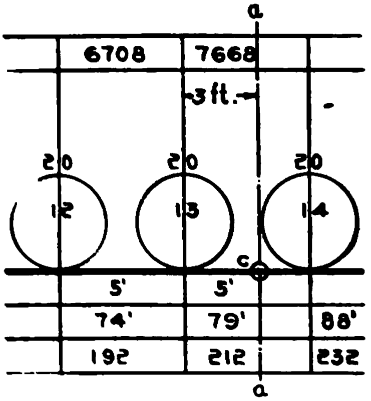


Fig. 86. Calculation of Moment at a Point under Engine Load.

$$\begin{aligned} M_{a-a} &= 7\,668 + 212 \times 3 \\ &= 7\,668 + 636 \\ &= 8\,304 = 8\,304\,000 \text{ pound-feet, there being no uniform load.} \end{aligned}$$

To illustrate the method when applied to points in the uniform load, assume the point to be 7 feet to the right of line 2. The position is illustrated in Fig. 87. The moment is:

$$M_{b-b} = 19\,304 + 304 \times 7 + \frac{7^2 \times 2}{2}.$$

$$21\,481 = 21\,481\,000 \text{ pound-feet.}$$

The use of the moment diagram is now apparent. Reactions due to any position of the engines may be determined by dividing the span into the value obtained for the moment at the right end of the span. Likewise, if the moment of the reaction about any panel point is determined and from it the moment of the wheel loads about that same panel point are subtracted, then the result, divided by the height of the truss, will give the chord stress. For example, if the right end of an 8-panel 196-foot span truss, height 25 feet, came 7 feet to the right of the vertical line 2, then the moment at this point (see Fig. 87) would be 21 481 000, and the reaction would be 21 481 000 ÷ 196 = 109 600. This position of the loads would cause the panel point L_6 to come 3 feet to the right of wheel 13. The moment of the reaction about L_6 is 109 600 × 6 × 24.5 = 16 111 200;

and the chord stress U_6L_6 for this position of the engine is:

$$\frac{16\,111\,200 - 8\,304\,000}{25} = -312\,000 \text{ pounds.}$$

In using the engine to determine the shear in any particular panel, it must be remembered that the shear is *not* the left reaction less all the loads to the left of the panel point on the right of the section, as the loads in the panel under consideration are carried on stringers, and these stringers transfer a portion of the loads to the panel point on the left of the panel, and a portion to the panel point on the right of the panel. Only that portion of the loads in the panel which is transferred to the left panel point should be subtracted from the reaction, as should all of the loads to the left of the panel under consideration. If, in a 6-panel 120-foot span Pratt truss, the wheel 6 comes at L_2, the left reaction will be:

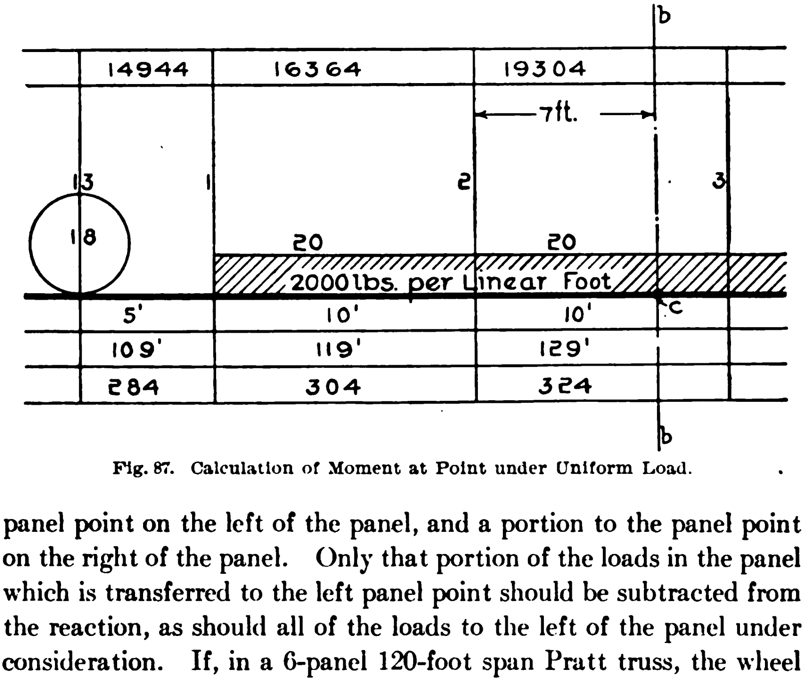


Fig. 87. Calculation of Moment at Point under Uniform Load.

$$R_1 = \frac{1}{120}\left(16\,364 + 3 \times 284 + \frac{3^2 \times 2.0}{2}\right) = 143.6;$$

and the loads in the first two panels will be in position as indicated by Fig. 88, the wheel 3 being 1 foot to the right of point L_1. Let it be required to determine the shear in the panel L_1L_2 when the loads are in this position. It will be the reaction 143.6 minus loads 1 and 2 and also that portion of the loads 3, 4, and 5 which will be transferred by the stringers to the point L_1. As the stringers are simple

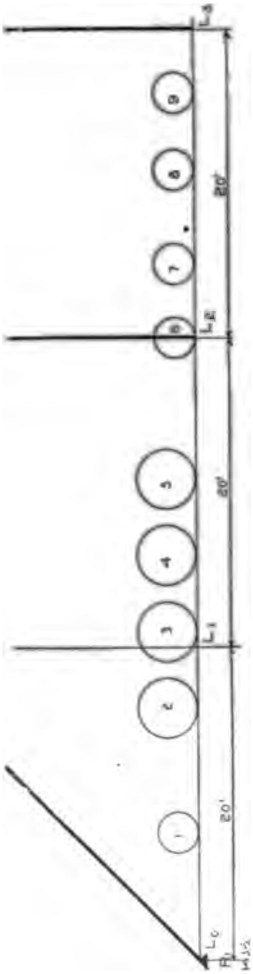


Fig. 88. Determination of Shear in Panel under Engine Load.

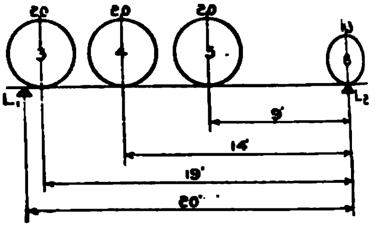


Fig. 89. Shear Diagram for Panel under Engine Load.

beams, the amount transferred to L_1 will be the reaction of the stringer L_1L_2. Referring to Fig. 89, the reaction is:

$$R_{L_1} = (20 \times 9 + 20 \times 14 + 20 \times 19) \div 20 = 42.0$$

The shear in the second panel is now found to be:

$$V_2 = 143.6 - (10 + 20 + 42.0) = +71.6$$

In the majority of cases where it is necessary to determine the shear in a panel, none of the loads will be in the panel to the left of the one under consideration. In this case the operation is somewhat simplified, as the engine diagram can be used directly. If the engine be placed so that the third wheel is at L_2, wheel 16 will be just over the right support, and the left reaction will be:

$$R_1 = 12\,041 \div 120 = 100.3.$$

As there are no wheel loads in the first panel, the amount to be subtracted from the reaction will be that proportion of the loads 1 and 2 which is transferred to L_1; and this (see Fig. 90) is $230 \div 20 = 11.5$. The shear in the second panel is then $100.3 - 11.5 = +88.8$.

From inspection of the resulting shear in the second panel when wheel 6 is at L_2 and when wheel 3 is at L_2, it is seen that different wheels at L_2 will give different shears in the panel to the left. Evidently there is some wheel which will give the greatest shear possible. The same is true of the relation between wheels and moments. The next two articles are devoted to subject-matter which will enable one to tell which of several wheels is the correct wheel at the point, without the necessity of solving for the shear each time every wheel is at the point.

45. **Position of Wheel Loads for Maximum Shear.** By methods of differential calculus, it can be proved that, for any system, either of wheel loads or wheel loads followed by a uniform load (see Fig. 91), the correct wheel that should be at the panel point b in order to give a very great or maximum shear in the panel $a - b$, is such a wheel that the quantity $Q = \frac{W}{m} - G$ is positive when $q = \frac{W}{m} - (G + P)$ is negative. In these equations,

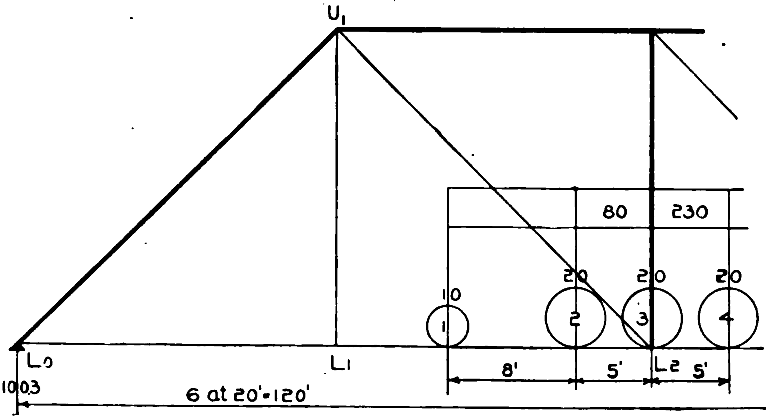


Fig. 90. Determination of Shear in Panel under Engine Load.

W = Total load on the truss;
m = Number of panels in the truss;
G = Load in panel under consideration; and
P = Load at panel points on right of panel.

If a load is directly over the panel point a, it is not to be included in the weight G; neither is P included in the weight G. If a wheel load should come directly over the right end of the truss, it should not be considered in the quantity W.

The only way to determine which wheel is the correct one, is to try wheel 1, then wheel 2, and so on, until the wheel or wheels are reached that will give the Q and q signs of an opposite character.

The process should not be stopped there, but the next succeeding wheels should be tried until Q and q again have the same sign.

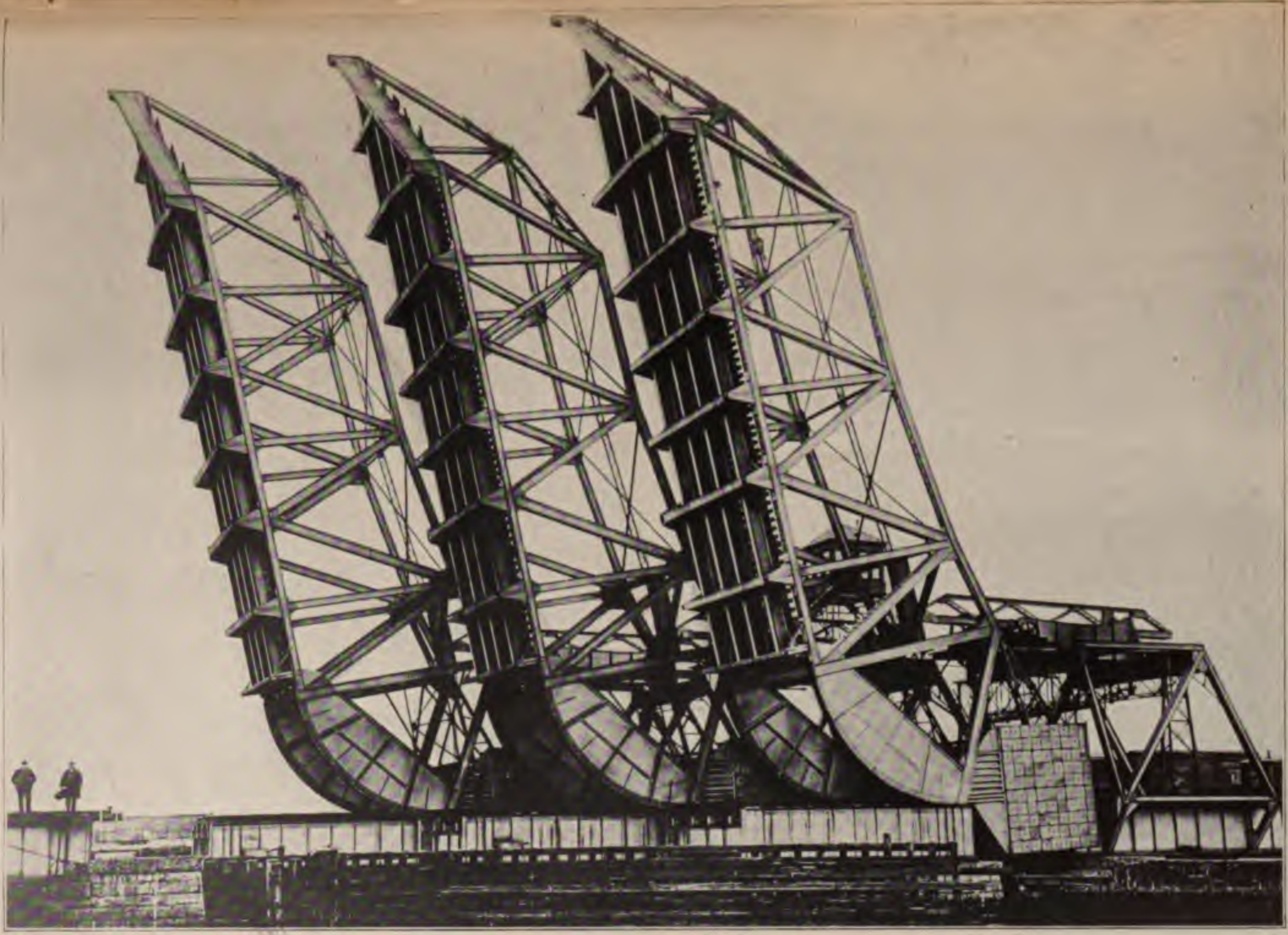
As an example, let it be required to determine the position of the wheel loads to produce the maximum positive shears in a 6-panel 120-foot Pratt truss. This work should be arranged in tabular form, and Table V is found to be convenient.

TABLE V

Determination of Position of Wheel Loads for Maximum Shear

$(m = 6)$

Panel Point	Wheel at Point	G	$\frac{W}{m}$	P	$G+P$	Q	q	Remarks
L_1	2	10	$\frac{284}{6} = 47.33$	20	30	+	+	
L_1	3	30	$\frac{292}{6} = 48.67$	20	50	+	−	gives a maximum
L_1	4	50	$\frac{302}{6} = 50.03$	20	70	+	−	gives a maximum
L_1	5	60	$\frac{302}{6} = 50.03$	20	100	−	−	
L_2	2	10	$\frac{232}{6} = 38.67$	20	30	+	+	
L_2	3	30	$\frac{245}{6} = 40.83$	20	50	+	−	gives a maximum
L_2	4	50	$\frac{258}{6} = 43.00$	20	70	−	−	
L_3	1	0	$\frac{152}{6} = 25.67$	10	10	+	+	
L_3	2	10	$\frac{172}{6} = 28.67$	20	30	+	−	gives a maximum
L_3	3	30	$\frac{192}{6} = 34.0$	20	50	+	−	gives a maximum
L_3	4	50	$\frac{212}{6} = 37.0$	20	70	−	−	
L_4	1	0	$\frac{116}{6} = 19.33$	10	10	+	+	
L_4	2	10	$\frac{129}{6} = 21.50$	20	30	+	−	gives a maximum
L_4	3	30	$\frac{142}{6} = 23.67$	20	50	−	−	
L_5	1	0	$\frac{70}{6} = 11.67$	10	10	+	+	
L_5	2	10	$\frac{90}{6} = 15.00$	20	30	+	−	gives a maximum
L_5	3	30	$\frac{103}{6} = 17.17$	20	50	−	−	



SIX-TRACK ROLLING-LIFT BRIDGE OVER FORT POINT CHANNEL, BOSTON, MASSACHUSETTS

On line of New York, New Haven & Hartford Railroad. Designed by the Scherzer Rolling-Lift Bridge Company, Chicago, Ill.

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A study of Table V shows the fact that wheel 1 can never produce a maximum. It also shows that there are in some cases two positions which will give large values of the moment. In these cases the shears for each position of the engines must be determined in order to tell which wheel at the panel point in reality gives the greatest. In practical work it is customary to use the first position found, as the difference in the shears resulting from the use of the two positions is not large enough to affect the final design.

Fig. 92 shows the engine diagram on the truss in the correct position to give the maximum shear in the second panel. The weight of wheel 16 is not included in the weight W, as it is directly over the right support.

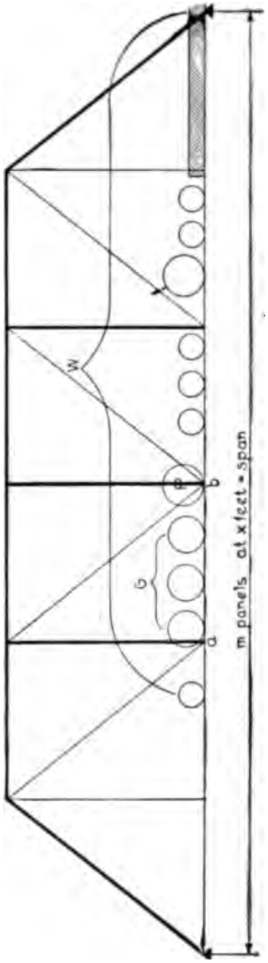


Fig. 91. Engine Diagram for Determining Position of Wheel Loads to Give Maximum Shear in Panel.

46. Position of Wheel Loads for Maximum Moments. In this case the methods of differential calculus are employed to determine which wheels will, if placed at a point, give a maximum moment at that point. For any system, either of wheel loads or of wheel loads followed by a uniform load, that wheel which will cause $K = \frac{Wn}{m} - L$ to be positive, and $k = \frac{Wn}{m} - (L + P)$ to be negative, is the wheel. Here n is the number of the panel under consideration, and is to be reckoned from the left end; L = the load to the left of the point under consideration; and the remainder of the letters signify the same as they do in Article 45. In some cases there will be more than one position of the loads which will satisfy the above condition. It is then necessary to work out the actual moments created by the loads in each position, in order to find out which is the largest. The

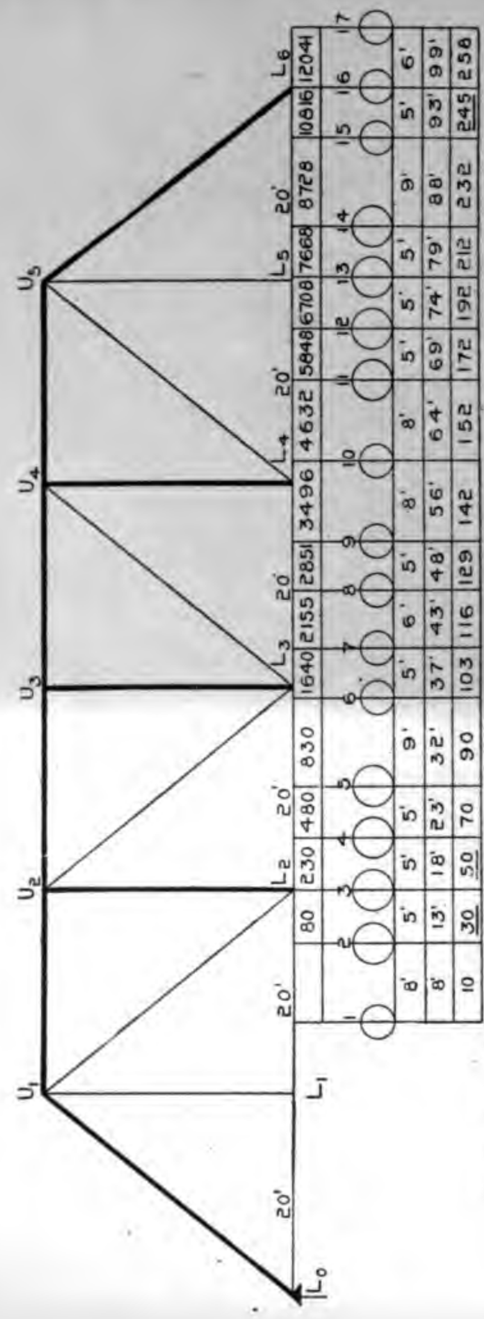


Fig. 92. Diagram of 6-Panel Pratt Truss Loaded with Engine in Position to Give Maximum Shear in Second Panel.

position of the loads for the greatest moments should be determined for all panel points except the one on the extreme right, as the greatest moment possible may be caused by wheels of the rear engine being on the point on the right-hand side of the truss, instead of the wheels of the front engine being at the corresponding point on the left-hand side.

In general, it may be said that there will be a number of wheels which, if placed at the panel point in the center of the span, will satisfy the given conditions. In this particular case, it is not necessary to determine all of the moments. The greatest moment possible will occur when that one of the heaviest wheels of the second locomotive which gives the heaviest load upon the truss is at the point. In case several of the heavy wheels give the same maximum load W, use the first wheel which gives this maximum W.

Let it be required to determine the position of the wheel loads for maximum moments at the lower panel points of the 6-panel 120-foot Pratt truss of Article 45. The necessary work can be conveniently arranged in the form of a table, as is done in Table VI.

TABLE VI

Determination of Position of Wheel Loads for Maximum Moments

PANEL POINT	WHEEL AT POINT	L	$\frac{Wn}{m}$	P	$L+P$	K	k	REMARKS
L_1	2	10	$284 \div 6 = 47.3$	20	30	+	+	
L_1	3	30	$292 \div 6 = 48.7$	20	50	+	−	gives a maximum
L_1	4	50	$302 \div 6 = 50.3$	20	70	+	−	gives a maximum
L_1	5	60	$302 \div 6 = 50.3$	20	80	−	−	wheel 1 is off bridge
L_2	5	70	$(271 \div 6) \times 2 = 90.3$	20	90	+	+	
L_2	6	90	$(290 \div 6) \times 2 = 96.7$	13	103	+	−	gives a maximum
L_2	7	103	$(300 \div 6) \times 2 = 100.0$	13	116	−	−	
L_3	8	116	$(271 \div 6) \times 3 = 135.5$	13	129	+	+	
L_3	9	129	$(284 \div 6) \times 3 = 142.0$	13	142	+	−	gives a maximum
L_3	10	142	$(298 \div 6) \times 3 = 149.0$	10	152	+	−	gives a maximum
L_3	11	142	$(304 \div 6) \times 3 = 151.3$	20	162	+	−	gives a maximum
L_3	12	142	$(294 \div 6) \times 3 = 147.0$	20	162	+	−	gives a maximum
L_3	13	142	$(284 \div 6) \times 3 = 142.0$	20	162	0	−	gives a maximum
L_3	14	142	$(274 \div 6) \times 3 = 137.0$	20	162	−	−	
L_4	11	152	$(271 \div 6) \times 4 = 180.6$	20	162	+	+	Note wheel 18 not included.
L_4	12	172	$(284 \div 6) \times 4 = 189.4$	20	192	+	−	gives a maximum
L_4	13	192	$(294 \div 6) \times 4 = 196.0$	20	212	+	−	gives a maximum
L_4	14	212	$(304 \div 6) \times 4 = 202.6$	20	232	−	−	

One should carefully note that in certain positions, as when wheels 11, 12, 13, and 14 are at L_3, some wheels are to the left of the left support; that is, they are not upon the bridge. In all such cases they are counted neither in the quantity L nor in W.

In the case of L_3, wheel 11, being the first large driver of the second engine, will give the greatest moment, as it is the first driver to come at the point when the maximum load of 304 000 pounds is on the truss. Fig. 93 shows the engine diagram on the truss in correct position to give the maximum moment at point L_2.

47. Pratt Truss under Engine Loads. In order to exemplify the use of the engine-load diagram, let it be required to determine the stresses in the Pratt truss of Article 45 due to E 40 loading, the

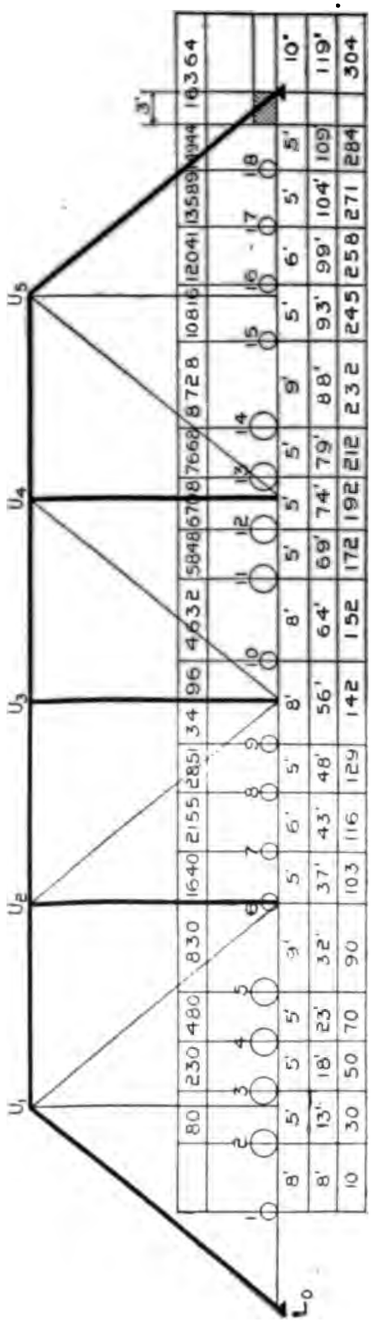


Fig. 93. Diagram of 6-Panel Pratt Truss Loaded with Engine in Position to Give Maximum Moment at Panel Point.

height being 25 feet. The secant is $(\overline{25}^2 + \overline{20}^2)^{\frac{1}{2}} \div 25 = 1.28$.

The maximum positive shears in the various panels should first be computed. These, written in reverse order, will be the maximum negative or minimum shears. Table V should now be referred to, and an outline diagram drawn to the same scale as the engine used, on which to place the engine diagram in the correct position. The various values can then be read off the diagram at the right-hand end of the truss. It will be found convenient to lay off to scale the first ten feet of the lower chord of the truss from the right support, making the divisions one foot apart. This will enable one to ascertain the distance of the last wheel load from the right support, or the amount of uniform load upon the bridge, without scaling or further computation. In case it is desired to have the wheel loads appear on the lower chord, as in Fig. 93, the outline of the truss should be on tracing cloth or transparent paper. This is not to be advised, however, as errors are likely to occur because of failure to distinguish clearly the various numerical values. It is far better to place the diagram as in Fig. 92, in which case the outline of both the truss and the diagram can be drawn on good stiff paper.

For wheel 3 at point L_1 (see Articles 44 and 45), the left reaction is as follows, there being four feet of uniform load upon the truss:

$$R_l = \left(16\,364 + 284 \times 4 + \frac{4^2 \times 2.0}{2}\right) \div 120 = 146.0;$$

and the proportion of loads in the panel which is transferred to the point L_0 by the stringers is $230 \div 20 = 11.5$. The shear is therefore $V_1 = +146.0 - 11.5 = +134.5$. The computation for the shear when wheel 4 is at the point, will not be made; for, as has been noted before, the result will not be much different from the above.

For wheel 3 at L_2, wheel 16 comes over the right support. The left reaction is:

$$R_l = 12\,041 \div 120 = 100.3;$$

the proportional part of the loads which is transferred to L_1 is 11.5; and the shear is:

$$V_2 = (+100.3 - 11.5) = +88.8.$$

For wheel 2 at L_3, wheel 11 comes four feet from the right support. The left reaction is:

$$R_l = (5\,848 + 172 \times 4) \div 120 = 54.5.$$

That part of wheel 1 which is transferred to L_2 is $80 \div 20 = 4.0$, and the shear is therefore:

$$V_3 = (+54.5 - 4) = +50.5.$$

For wheel 2 at L_4, wheel 9 comes over the right support. The left reaction is:

$$R_l = 3\,496 \div 120 = 29.1.$$

That part of wheel 1 which is transferred to L_3 is 4.0, and the shear is therefore:

$$V_4 = (+29.1 - 4.0) = +25.1.$$

For wheel 2 at L_5, wheel 5 is five feet from the right support, and the left reaction is:

$$R_l = (830 + 90 \times 5) \div 120 = 10.7$$

and the shear is:

$$V_5 = (+10.7 - 4.0) = +6.7.$$

If the dead panel load is 20 000 pounds, all the shears may now be written as follows:

DEAD-LOAD V	+ LIVE-LOAD V	− LIVE-LOAD V
$V_1 = +50.0$	$+134.5$	$\pm\ 0.0$
$V_2 = +30.0$	$+\ 88.8$	$-\ 6.7$
$V_3 = +10.0$	$+\ 50.5$	-25.1
$V_4 = -10.0$	$+\ 25.1$	-50.5
$V_5 = -30.0$	$+\ 6.7$	-88.8
$V_6 = -50.0$	$\pm\ 0.0$	-134.5

A comparison of the shears in the third and fourth panels shows that counters are required. The stress in these counters is:

$$U_3L_2 = U_3L_4 = +1.28 \times (25.1 - 10.0) = +19.2$$

As it is known that positive shears cause a compressive stress in L_0U_1 and tensile stresses in the diagonals, and that negative shears produce compressive stresses in the intermediate posts, the left half of the bridge being considered, the web stresses for dead and live load can be determined without in all cases writing the stress equations in order to determine the sign. It should be remembered that one-third of the dead panel load, or 6 700 pounds, is applied at the panel points of the top chord.

Dead-Load Stresses in the Diagonals—

$$L_0U_1 = -1.28 \times 50 = -64.0$$
$$U_1L_2 = +1.28 \times 30 = +38.4$$
$$U_2L_3 = +1.28 \times 10 = +12.8$$

Dead-Load Stresses in the Verticals. For U_2L_2, the section passed will cut U_1U_2, U_2L_2, and L_2L_3, and the shear on this section will be $50 - 2 \times 13.3 - 6.7 = +16.7$. The stress equation is $+16.7 + U_2L_2 = 0$, from which $U_2L_2 = -16.7$.

The dead-load stress in U_3L_3 is found by passing a circular section around U_3. Then $-U_3L_3 - 6.7 = 0$, from which $U_3L_3 = -6.7$. In a similar manner, by passing a section around L_1, the stress is found to be $+13.3$.

Live-Load Stresses in the Diagonals—

	MAXIMUM	MINIMUM
L_0U_1	$= -1.28 \times 134.5 = -172.2$	0
U_1L_2	$= +1.28 \times 88.8 = +113.6$	$-1.28 \times 6.7 = -8.6$
U_2L_3	$= +1.28 \times 50.5 = +64.7$	0

Live-Load Stresses in the Verticals. The maximum stress in U_1L_1 occurs when one of the large drivers is at L_1, and the loads in the first panel are as near as possible one-half the sum of the loads in panels 1 and 2 and the load at L_1. This can be established as a fact by use of the differential calculus. In the present case, this condition is satisfied when wheel 4 is at L_1. Then the weight of the wheels in panel 1 is 50 000 pounds, and the sum total is 116 000 pounds. If wheel 13 be placed at L_1, the result will be the same, and then the engine diagram can be used. Fig. 94 represents the engine diagram in place, ready to use. According to Article 44, the value 480 is the moment of wheels 10 to 12 about L_1. Therefore $480 \div 20$ (20 is the panel length) $= 24.00$, is that amount of wheels 10 to 12 which is transferred to L_0. In like manner, $529 \div 20 = 26.45$ is the amount of wheels 14 to 16 transferred to L_2. As the total weight of the loads in the two panels is 116 000 pounds, the amount transferred to L_1 must be $116.0 - (24.00 + 26.45) = 65.55$, and the stress in U_1L_1 is therefore $+65.55$.

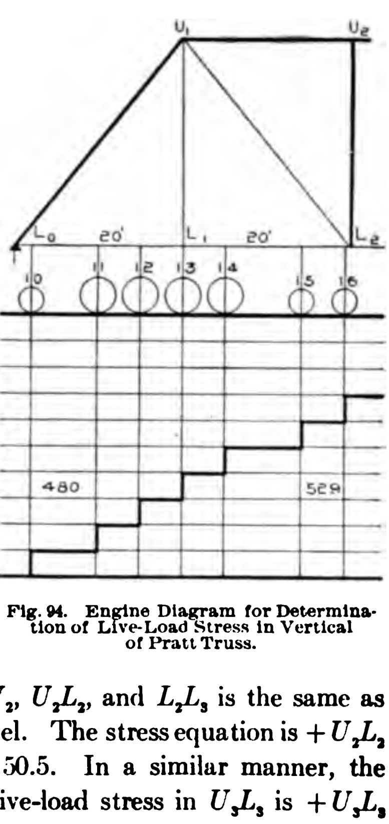


Fig. 94. Engine Diagram for Determination of Live-Load Stress in Vertical of Pratt Truss.

The maximum live-load stress in L_2U_2 occurs when the loading is in a position to give the maximum shear in the third panel, as the shear at a section cutting U_1U_2, U_2L_2, and L_2L_3 is the same as that at a vertical section in the panel. The stress equation is $+U_2L_2 + 50.5 = 0$, from which $U_2L_2 = -50.5$. In a similar manner, the stress equation for the maximum live-load stress in U_3L_3 is $+U_3L_3 + 25.1 = 0$, as U_3L_4 is working, and therefore $U_3L_3 = -25.1$. As in the case of the analysis of the Pratt truss under uniform load (see Article 39), the dead-load stress of -6.7 cannot be added to this stress of -25.1 to obtain the maximum; but the dead-load stress in U_3L_3 must be obtained when diagonals U_3L_3 and U_3L_4 are in action. In the manner explained in Article 39, this is found to be $+3.30$.

It will be found that as the engines come on the bridge from the left, the counters come into action in the case of U_2L_2; and in the case of U_3L_3, both U_2L_3 and L_3U_4 act, thus causing the live-load stress in these verticals to be zero; and when this is the case, the dead-load stress is $-$ 6.7, which is the minimum.

Dead-Load Chord Stresses. The dead-load chord stresses can be found by any of the methods previously given; but they will be found by the tangent method as indicated below, the tangent being $20 \div 25 = 0.8$:

$$\begin{aligned}
L_0L_1 &= +0.8 \times 50 = +40.0 = L_1L_2 \\
U_1U_2 &= -(50 + 30) \times 0.8 = -64.0 \\
U_2U_3 &= -(50 + 30 + 10) \times 0.8 = -72.0 \\
L_2L_3 &= -U_1U_2 = -(-64.0) = +64.0
\end{aligned}$$

Live-Load Chord Stresses. On account of the wheel loading, no ratio can be established between these stresses and the dead-load chord stresses. The maximum moments at each point must be determined, and these divided by the height of the truss will give the chord stresses. For all points to the left of the center of the bridge, the main diagonal will act. For points to the right of the center, an uncertainty exists. The shear in the panels on either side of the point under consideration should be determined when the loading is in position to give the maximum moment at that point. This will indicate which diagonals act, which fact will indicate for what chord member that point is the center of moments.

When wheel 3 is at L_1, four feet of uniform load are on the truss, and the left reaction is:

$$R_1 = (16\,364 + 384 \times 4 + \frac{4^2 \times 2}{2}) \div 120 = 146.0.$$

The moment of this reaction about L_1, less the moment of wheels 1 and 2 about L_1, will be the moment at L_1 due to this loading. The moment of wheels 1 and 2 about L_1 is taken from the diagram, where it occurs in the first line of values just to the right of the vertical line through wheel 3, and therefore:

$$M_1 = 146.0 \times 20 - 230 = 2\,690\,000 \text{ pound-feet.}$$

When wheel 4 is at L_1, there are nine feet of uniform load on the truss, and the left reaction is:

$$R_1 = (16\,364 + 284 \times 9 + \frac{9^2 \times 2}{2}) \div 120 = 158.0;$$

and in this case,

$$M_1 = 158.0 \times 20 - 480 = 2\,680\,000 \text{ pound-feet.}$$

As this is less than when wheel 3 is at the point, wheel 3 gives the greatest moment.

When considering the point L_2 with wheel 6, the left reaction is:

$$R_l = (16\,364 + 284 \times 3 + \frac{3^2 \times 2}{2}) \div 120 = 143.5$$

$$M_2 = 143.5 \times 2 \times 20 - 1\,640$$

$$= 4\,100\,000 \text{ pound-feet.}$$

The conditions at L_3 indicate that there are several wheels which give large moments; but according to Article 46, wheel 11 gives the maximum moment. When this wheel is at L_3, wheel 1 is off the truss, and 15 feet of uniform load are on the truss. The moment of *all* the loads about the *right support* is $19\,304 + 304 \times 5 + \frac{5^2 \times 2}{2} = 20\,849$, from which should be subtracted the moment of wheel 1 about the right support. This moment of wheel 1 is $10 \times 124 = 1\,240$, and the moment about L_6 of all loads on the truss is $20\,849 - 1\,240 = 19\,609$. The left reaction is:

$$R_l = 19\,609 \div 120 = 163.4$$

$$M_3 = 163.4 \times 3 \times 20 - (5\,848 - 10 \times 64)$$

$$= 4\,596\,000 \text{ pound-feet.}$$

In the case of L_4, the reactions and the moments for the two positions are:

For wheel 12, $R_l = 16\,364 \div 120 = 136.4$

$$M_4 = 136.4 \times 4 \times 20 - 6\,708$$

$$= 4\,204\,000 \text{ pound-feet.}$$

For wheel 13, $R_l = (16\,364 + 5 \times 284 + \frac{5^2 \times 2}{2}) \div 120 = 148.4$

$$M_4 = 148.4 \times 4 \times 20 - 7\,668$$

$$= 4\,204\,000 \text{ pound-feet,}$$

which shows that each wheel gives the same moment, and also that the moment is greater than that at L_2, the corresponding point on the left-hand side of the center of the truss. As L_2 is the center of moments of U_1U_2, then, if the center of moments for U_4U_5 falls at L_4 (that is, if L_4U_5 acts), the stress in U_4U_5 will be greater than the stress in U_1U_2 when wheel 6 is at L_2. Of course, if the engine came on the truss from the left, U_1U_2 would receive the same stress that U_4U_5 now receives. According to the shears, L_4U_5 always acts, and therefore the center of moments for U_4U_5 does fall at L_4.

TABLE VII

Stresses in a Pratt Truss

Kind of Stress	End-Post	Verticals			Diagonals			Upper Chord		Lower Chord		
	L_0U_1	U_1L_1	U_2L_2	U_3L_3	U_1L_2	U_2L_3	L_2U_3	U_1U_2	U_2U_3	L_0L_1	L_1L_2	L_2L_3
Dead-Load. Live-Load..	− 64.0 − 172.2 0.0	+ 13.3 + 65.6 0.0	− 16.7 − 50.5 0.0	{ − 6.7 { + 3.3 − 25.1 0.0	+ 38.4 + 118.6 − 8.6	+ 12.8 + 64.7 0.0	− 12.8 0.0 + 32.0	− 64.0 − 16[illegible]	− 72.0 − 1[illegible]3.8	+ 40.0 + 107.5	+ 40.0 + 107.5	+ 64 0 + 168.2
Maximum.. Minimum..	− 236.2 − 64.0	+ 78.9 + 13.3	− 67.2 − 16.7	− 21.8 − 6.7	+ 157.0 + 29.8	+ 77.5 0.0	+ [illegible] [illegible]	[illegible] [illegible]	[illegible]5.8 [illegible]2.0	+ 147.5 + 40.0	+ 147.5 + 40.0	+ 232.2 + 64.0

The various moments are written in order, as such action will facilitate the remainder of the computations.

$$M_1 = 2\,690$$
$$M_2 = 4\,100$$
$$M_3 = 4\,596$$
$$M_4 = 4\,204$$

The chord stresses are now found to be:

$$[illegible]_0L_1 = L_1L_2 = + \frac{2\,690}{25} = +107.5$$

$$U_1U_2 = - \frac{4\,100}{25} = -164.0$$

$$U_2U_3 = - \frac{4\,596}{25} = -183.8$$

$$U_4U_5 = - \frac{4\,204}{25} = -168.2$$

$$[illegible]L_3 = -U_1L_2 = -(-164.2) = +164.2$$

$$[illegible]L_4 = -U_4U_5 = -(-168.2) = +168.2$$

[illegible]hen the load comes on from the [illegible]t, the stresses in U_1U_2 and L_2L_3 will be -168.2 and $+168.2$ respectively, which are the maximum live-load stresses for these members.

Instead of placing the values of the stresses on a truss outline, they are sometimes put in tabular form, as in Table VII.

48. **Impact Stresses.** When an engine is at rest on a bridge, the stresses in the members are the same as those computed for that loading. When the loads move across the bridge at any speed, the vibrations and the shocks produced by the counterweights in the drivers and by other causes create stresses in the various members in excess of those computed by aid of the engine diagram. The excess stresses

are designated as *impact stresses.* This term, however, is misleading to a certain extent, as causes other than the impact or pounding of the engine wheels help to produce the stresses referred to.

It is a well-known fact capable of mathematical demonstration, that a load, if suddenly applied, will produce a stress equal to twice that which it will produce as a static load; also, that as the ratio of the weight of the load to the weight of the structure increases, the vibrations produced by the impact will be less. These two facts are the basis of most of the empirical formulæ for impact stresses; and empirical formulæ are used to obtain these stresses, as the existing conditions and producing causes are not such as to make them susceptible of mathematical treatment. The result of experiments on actual bridges under the effect of passing engines and trains, have been the basis of many formulæ. One of these is:

$$I = S\,(\frac{300}{L + 300}),$$

where I = Impact stress in the member;
S = Live-load stress in the member caused by the engine load when at rest;
L = Length of that part of the bridge which is loaded when the stress S is produced; and
300 = A constant value derived from experiments.

This formula was proposed by C. C. Schneider in 1887, and is given in the "Transactions" of the American Society of Civil Engineers, Vol. 34, page 331. While it does not take into consideration the relative weights of the bridge and the live-load loads, this formula does make allowance for the time it takes to produce the stress, by introducing L, the distance over which the engine passes before causing the stress S. It is seen that the smaller the distance L, the greater will be the impact stress for any given value of S. When L becomes exceedingly small, the effect would be that of a suddenly applied load, and the impact stress would equal the stress S. Table VIII gives the values of $\frac{300}{L + 300}$, which is called the *impact coefficient,* for different values of L. Values not given may be interpolated.

For example, by consulting Fig. 92, which gives the position of the engines for the maximum live-load stress in U_1L_2, it is seen that 93 feet (the distance from wheel 1 to the right support) is the loaded

TABLE VIII

Values of the Impact Coefficient

L	$\frac{300}{L+300}$	L	$\frac{300}{L+300}$	L	$\frac{300}{L+300}$	L	$\frac{300}{L+300}$	L	$\frac{300}{L+300}$
5	0.984	31	0.906	57	0.840	83	0.783	145	0.674
6	0.980	32	0.904	58	0.838	84	0.781	150	0.667
7	0.977	33	0.901	59	0.836	85	0.779	155	0.659
8	0.974	34	0.898	60	0.833	86	0.777	160	0.652
9	0.971	35	0.896	61	0.831	87	0.775	165	0.645
10	0.968	36	0.893	62	0.829	88	0.773	170	0.638
11	0.965	37	0.890	63	0.826	89	0.771	175	0.632
12	0.962	38	0.888	64	0.824	90	0.769	180	0.625
13	0.958	39	0.885	65	0.822	91	0.767	185	0.619
14	0.955	40	0.882	66	0.820	92	0.765	190	0.612
15	0.952	41	0.880	67	0.817	93	0.763	195	0.606
16	0.949	42	0.877	68	0.815	94	0.761	200	0.600
17	0.946	43	0.875	69	0.813	95	0.759	210	0.588
18	0.943	44	0.872	70	0.811	96	0.758	220	0.577
19	0.940	45	0.870	71	0.809	97	0.756	230	0.566
20	0.937	46	0.867	72	0.806	98	0.754	240	0.556
21	0.935	47	0.865	73	0.804	99	0.752	250	0.546
22	0.932	48	0.862	74	0.802	100	0.750	260	0.536
23	0.929	49	0.860	75	0.800	105	0.741	270	0.526
24	0.926	50	0.857	76	0.798	110	0.732	280	0.517
25	0.923	51	0.855	77	0.796	115	0.725	290	0.508
26	0.920	52	0.852	78	0.794	120	0.714	300	0.500
27	0.917	53	0.850	79	0.792	125	0.706	400	0.429
28	0.915	54	0.847	80	0.789	130	0.698	500	0.375
29	0.912	55	0.845	81	0.787	135	0.690	600	0.333
30	0.909	56	0.843	82	0.785	140	0.682		

length. From Table VII, it is seen that the stress in U_1L_2 produced by this loading is +118.6; and from Table VIII, the impact coefficient for 93 feet is found to be 0.763. The impact stress is now computed:

$$I = 0.763 \times 118.6 = +90.6.$$

The maximum stress in U_1L_2 is now:

$$\begin{aligned} \text{Dead-load} &= +\ \ 38.4 \\ \text{Live-load} &= +118.6 \\ \text{Impact} &= +\ \ 90.6 \\ \hline \text{Maximum} &= +247.6 \end{aligned}$$

Table IX gives the necessary information for computing the impact stresses, and also gives the impact stresses corresponding to the maximum live-load stresses in the members of the truss of Article 47.

TABLE IX

Impact Stresses in a Pratt Truss

MEMBER	S	L	$\frac{300}{L+300}$	I	REMARKS
L_0U_1	-172.2	113	0.727	-125.2	4 ft. of uniform load on truss
U_1L_1	+ 65.6	37	0.890	+ 58.4	See succeeding text.
U_2L_2	- 50.5	68	0.815	- 41.3	Same as for U_2L_3
U_3L_3	- 25.1	48	0.862	- 21.6	Same as for U_3L_4
U_1L_2	+118.6	93	0.763	+ 90.6	Wheel 16 at L_6
U_2L_3	+ 64.7	68	0.815	+ 52.7	Wheel 11 is 4 ft. from L_6
U_3L_4	+ 32.0	48	0.862	+ 27.6	Wheel 9 at L_6.
U_1U_2	-168.2	114	0.724	-121.8	Wheel 13 at L_4. 5 ft. of uniform load on bridge.
U_2U_3	-183.8	114	0.724	-133.2	15 ft. of uniform load on bridge. Wheel 1 off bridge.
L_0L_2	+107.5	113	0.727	+ 78.2	Same loading as for L_0U_1
L_2L_3	+168.2	114	0.724	+121.8	Same loading as for U_2U_3

In the case of U_1L_1, it should be noted that only the wheels 10 to 16 inclusive cause the stress (see Fig. 94), and that the loaded length is the distance from wheel 10 to wheel 16.

Some specifications do not call for impact stresses. The unit-stresses in these specifications are made low, and the sections designed are large enough to withstand the additional stresses due to impact. In cases where the impact stresses are required, they must be considered in computing the maximum and minimum stresses.

49. Snow-Load Stresses. In some localities the snowfall is considerable, and its weight should be taken into account in computing stresses. This should be done by considering it as an additional dead load of 15 pounds per square foot of floor surface for every foot of snowfall. As it covers the entire floor surface, the stresses will be proportional to the dead-load stresses. Also it is evident that the snow load should not be taken into account in railroad bridges unless they have solid floors, as most of it falls through the open spaces between ties and stringers.

As an example, let it be required to determine the snow-load stresses in a member of a highway bridge, the dead-load stress in the member being +84.0, the dead panel load being 12 000 pounds, and the snow being $1\frac{1}{2}$ feet deep on the roadway, which is 14 feet wide. The snow panel load is:

$$\frac{1}{2}(14 \times 15 \times 1\frac{1}{2} \times 20) = 3\,150 \text{ pounds.}$$

In the above equation, 14 is the width of roadway; 15 is the weight in pounds of one square foot of snow one foot deep; and 20 is the length of one panel. One-half of the weight of snow must be taken, as half is carried by each truss. The snow-load stress is then:

$$\frac{3\,150}{12\,000} \times 84.0 = +22.1.$$

In like manner, all snow-load stresses can be computed.

Most of the standard specifications which have been published do not specify snow loads; and in fact it is not customary to include the snow load in any designs except those for bridges in extreme northern latitudes. It is hardly probable that the greatest load will come upon a country bridge when it is covered with snow. Also, in cities, the sidewalks are cleaned of snow; and so is the roadway if the city is of large size.

WIND-LOAD EFFECTS

50. Top Lateral System Through-Bridges. The unit-loads for this system are given in Article 26. Common practice is to take 150 pounds per linear foot of top chord, the end-post being considered part of the top chord in this computation.

In many of the longer-span modern bridges, the diagonals of this system are designed to take either tension or compression; but in the majority of the shorter spans, 200 feet and under, while generally consisting of angles or other stiff shapes, they are designed to take tension only. The verticals or top lateral struts take compression. This combination of tension diagonals and compression verticals makes the so-called *Pratt system of webbing:* and indeed the lateral systems, both top and bottom, are Pratt trusses in a horizontal position. Fig. 95 shows the side elevation of the truss of Article 47, and also the top and bottom laterals. The diagonals shown in full lines act when the wind is *right,* and those shown by dotted lines act when the wind is *left.* Wind *right* indicates that the wind is blowing from the right hand when a person stands facing the right end of

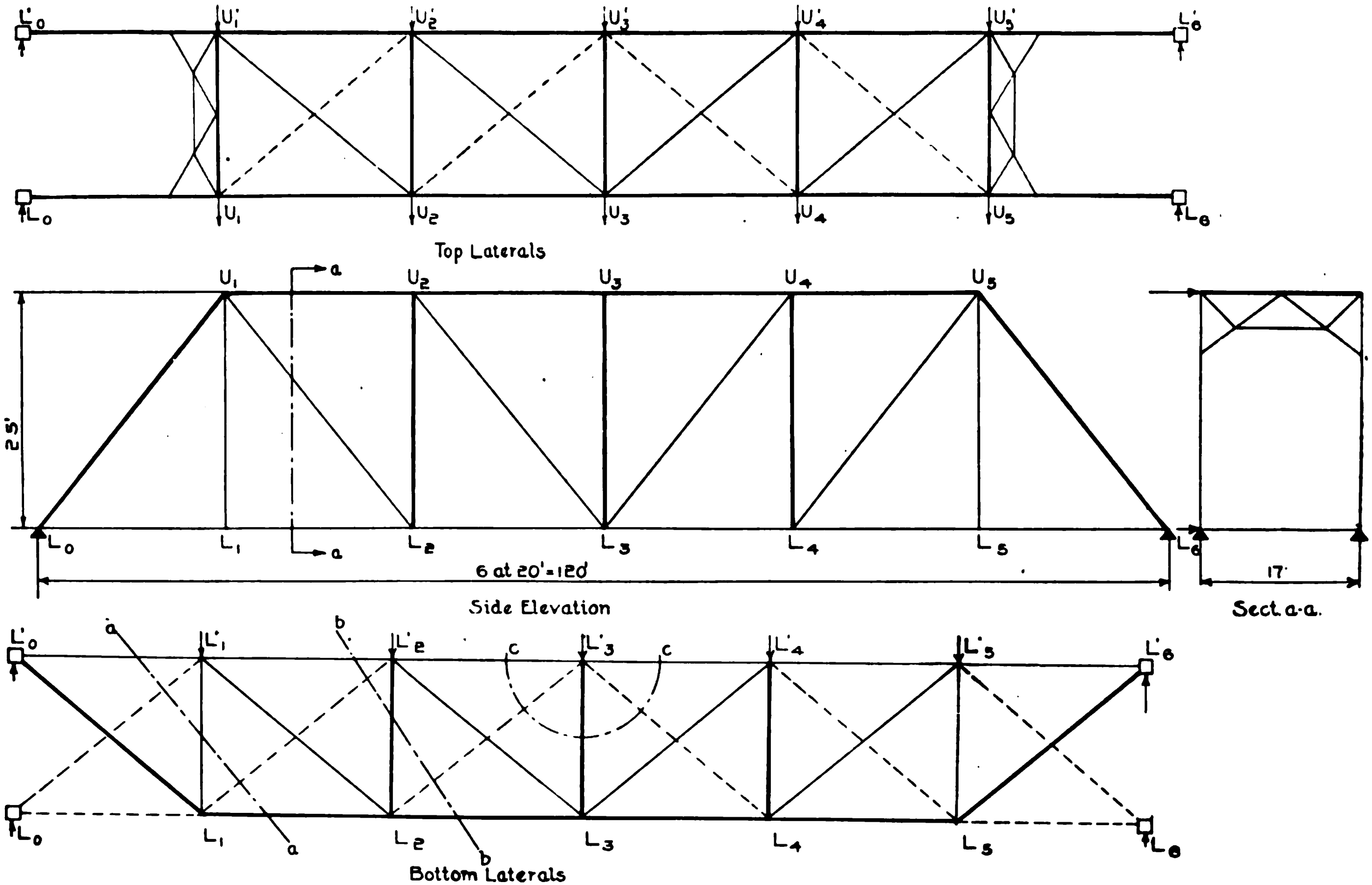


Fig. 95. Side and End Elevations and Top and Bottom Laterals of 6-Panel Pratt Truss.

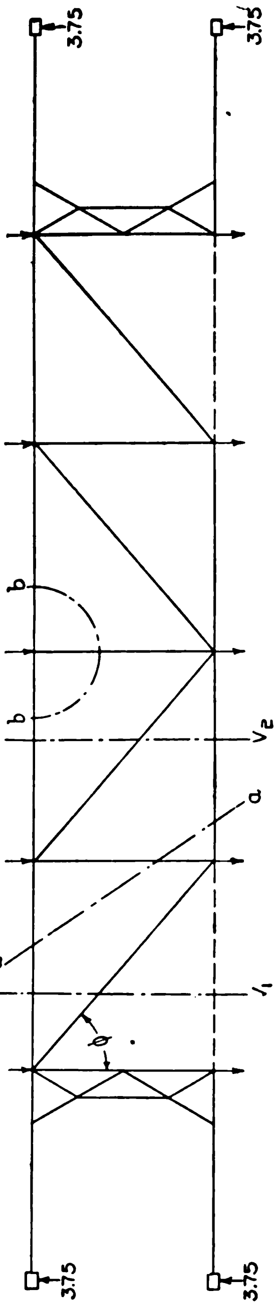


Fig. 96. Diagram Showing Distribution of Loads and Reaction in Pratt Truss of Fig. 95.

the bridge. Wind *left* indicates that the wind blows from a person's left when standing as above described.

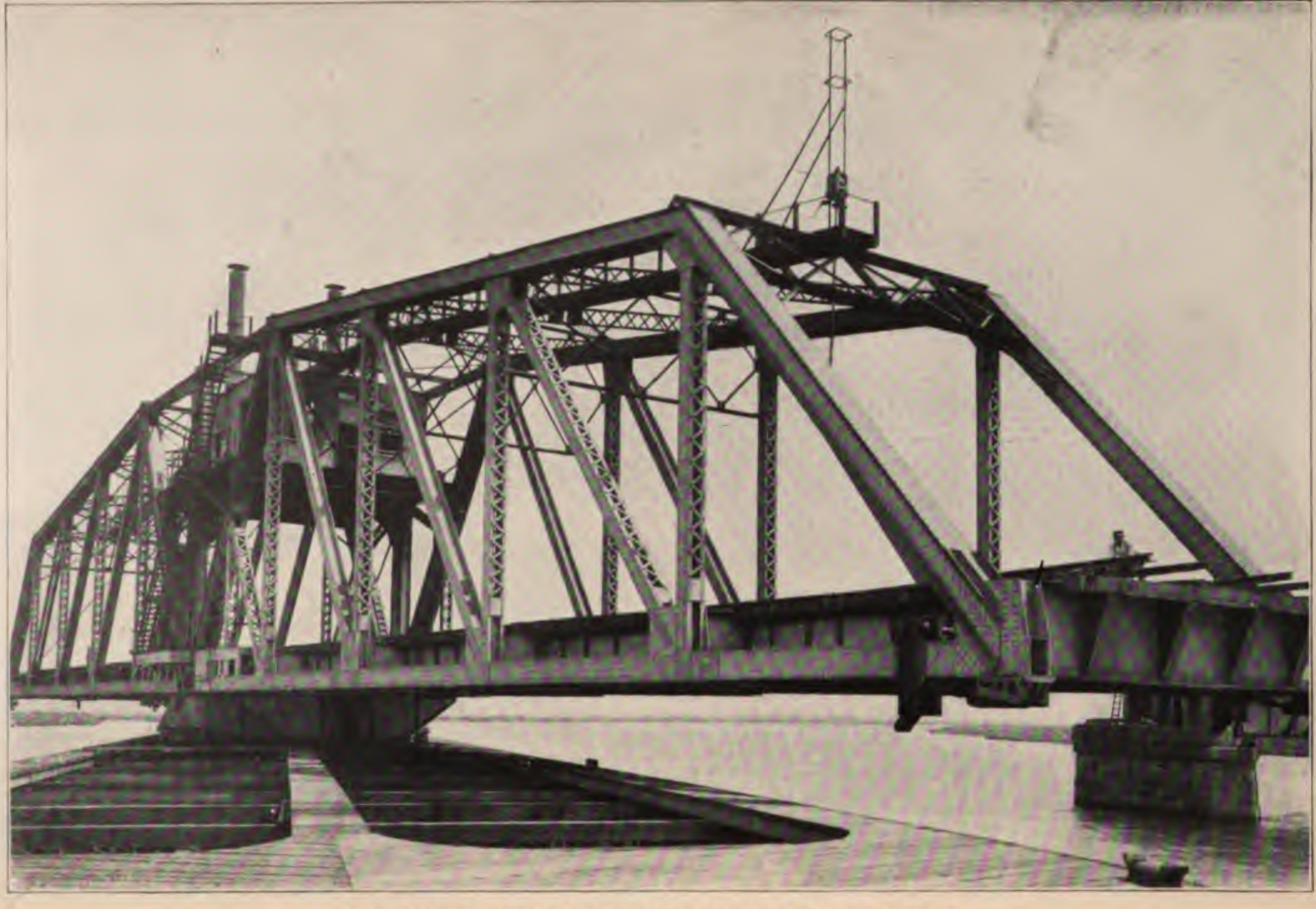
The wind load of 150 pounds is divided between the two trusses, this being exact enough for practical purposes; for, by actual experiment, the difference between the readings of wind-pressure gauges placed at points opposite each other in the top chords of a through-bridge was only from 8 to 10 per cent.

The problem, then, is one of a deck Pratt truss with a dead panel load of 150 × 20 = 3.0 divided between the two chords. Fig. 96 shows the distribution of loads and the reaction, it being considered that the portal bracings and the end-posts (see Fig. 95) are stiff enough to distribute the reaction equally between the bearing points L_0, L_0', L_6, L_6'. Each panel load is indicated by an arrow, and is equal to 3.0 ÷ 2 = 1.5. The reaction at each of the points L_0, L_0', L_6, and L_6' is 10 × 1.5 ÷ 4 = 3.75. The truss being symmetrical, the stresses in like members on each side of the center will be the same. The shears in the top system are:

$$V_1 = +2 \times 3.75 - 2 \times 1.5 = +4.5$$
$$V_{a\text{-}a} = +2 \times 3.75 - 3 \times 1.5 = +3.0$$
$$V_2 = +2 \times 3.75 - 4 \times 1.5 = +1.5$$

and the secant ϕ_1 is $(17^2 + 20^2)^{\frac{1}{2}} \div 17 = 1.544$. The stresses in the diagonals are:

$$U_1'U_2 = +1.544 \times 4.5 = +6.95$$
$$U_2'U_3 = +1.544 \times 1.5 = +2.32.$$



BRIDGE OVER THE PASSAIC RIVER AT NEWARK, NEW JERSEY

On line of Pennsylvania Railroad. Built in 1901. Consists of the following double-track spans: One riveted draw, 277 ft.; one deck plate-girder, 80 ft.; and four deck plate-girder spans, each 100 ft.

The vertical $U_2'U_2 = -3.0$; and by passing a section $b - b$ around U_3', the stress in $U_3'U_3$ is found to be -1.5.

In obtaining the chord stresses in this system, the case is the same as if the reactions were applied at U_1' and U_5', as the portal and end-posts are not in the same plane as the lateral system. The tangent method is the simplest to use in this case. The tangent is $20 \div 17 = 1.176$, and the stresses (see Fig. 95) are:

$$U_1'U_2' = -4.5 \times 1.176 = -5.29$$
$$U_2'U_3' = -(4.5 + 1.5) \times 1.176 = -7.06$$
$$U_1U_2 = 0$$
$$U_2U_3 = -U_1'U_2' = +5.29$$

Fig. 97 is a diagram with the stresses caused by wind right and wind left indicated thereon. The stresses for wind left can easily be written by inspection.

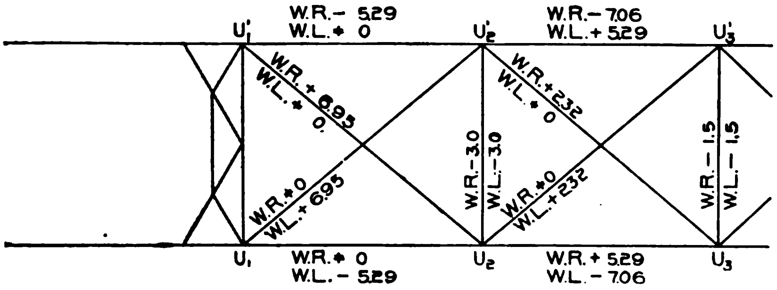


Fig. 97. Wind Stress Diagram of Pratt Truss of Fig. 95.

51. **Bottom Lateral Bracing, Through-Bridges.** Fig. 95 shows the lower lateral system with the panel points loaded with the fixed or dead wind load. In this case it is all taken as acting on one side, it being assumed that the floor system protects the leeward truss. The problem then becomes that of determining the stresses in a deck Pratt truss of 6 panels of 20 feet each, the height being 17 feet. When wind is *right*, the members shown by broken lines in Fig. 95 do not act.

The fixed wind load (Article 26) is 150 pounds per linear foot of chord. The panel load will be the same as before, 3.0, but all will be on one chord. The shears are:

$$V_1 = 2\tfrac{1}{2} \times 3.0 = +7.5$$
$$V_{a\text{-}a} = +7.5$$
$$V_2 = +7.5 - 3.0 = +4.5$$
$$V_{b\text{-}b} = +4.5$$
$$V_3 = +7.5 - 2 \times 3.0 = +1.5$$

The secant being 1.544, as previously computed, the web stresses are:

$$\begin{aligned} L_0'L_1 &= +7.5 \times 1.544 = +11.60 & L_1'L_1 &= -7.5 \\ L_1'L_2 &= +4.5 \times 1.544 = +\ 6.95 & L_2'L_2 &= -4.5 \\ L_2'L_3 &= +1.5 \times 1.544 = +\ 2.32 & L_3'L_3 &= -3.0 \end{aligned}$$

The stress in $L_3'L_3$ is determined by passing section $c - c$ and resolving the vertical forces at L_3' (see Fig. 95).

By using the tangent method, the chord stresses are computed as follows:

$$\begin{aligned} L_0'L_1' &= -7.5 \times 1.176 = -8.82 \\ L_1'L_2' &= -(7.5 + 4.5) \times 1.176 = -14.12 \\ L_2'L_3' &= -(7.5 + 4.5 + 1.5) \times 1.176 = -15.88 \\ L_1L_2 &= -L_0'L_1' = -(-8.82) = +8.82 \\ L_2L_3 &= -L_1'L_2' = -(-14.12) = +14.12 \end{aligned}$$

The wind load acting on the train is 450 pounds per linear foot. It is evident that the train may cover the span either partially or entirely, and therefore its action on the lower lateral system is the same as if it were stressed by a live load of 450 pounds per linear foot of truss.

The live panel load is 450 × 20 = 9.0. The maximum live-load reaction is 5 × 9.0 ÷ 2 = 22.5, and the positive live-load shears are:

$$\begin{aligned} V_1 &= +22.5 \\ V_2 &= (1 + 2 + 3 + 4)\frac{9.0}{6} = +15.0 \\ V_3 &= (1 + 2 + 3)\frac{9.0}{6} = +9.0 \end{aligned}$$

It is unnecessary to go further than the center, as only the maximum stresses are required in the members. The web stresses are computed as given below:

$$\begin{aligned} L_0'L_1 &= +22.5 \times 1.544 = +34.75 & L_1'L_1 &= -22.5 \\ L_1'L_2 &= +15.0 \times 1.544 = +23.15 & L_2'L_2 &= -15.0 \\ L_2'L_3 &= +\ 9.0 \times 1.544 = +13.91 & L_3'L_3 &= -\ 9.0 \end{aligned}$$

The maximum chord stresses due to this load of 450 pounds per linear foot of train, occur when the train covers the entire span; and they are directly proportional to the stresses produced by the fixed load, in the same ratio as the live panel load is to the fixed panel load. This ratio is $\frac{9.0}{3.0} = 3.0$. The chord stresses, therefore, are:

$L_0'L_1' = -8.82 \times 3 = -26.46$

$L_1'L_2' = -14.12 \times 3 = -42.36$

$L_2'L_3' = -15.88 \times 3 = -47.64$

$L_1L_2 = +8.82 \times 3 = +26.46$

$L_2L_3 = +14.12 \times 3 = +42.36$

Table X, Article 53, gives the stresses in the top and bottom lateral systems for wind right and wind left.

52. Overturning Effect of Wind on Truss. When the wind blows on the top chord, it tends to overturn the truss. As the truss is held down by its own weight, the action of the wind does not overturn it, but causes the dead-load reaction on the windward side to be less and that on the leeward side to increase by a like amount. The amount is $\pm V = \frac{\Sigma w}{2} \times \frac{h}{b}$, where Σw = the sum of all the wind panel loads, h = the height of the truss, and b = the distance center to center of trusses. The effect upon the leeward truss is the same as if two ver-

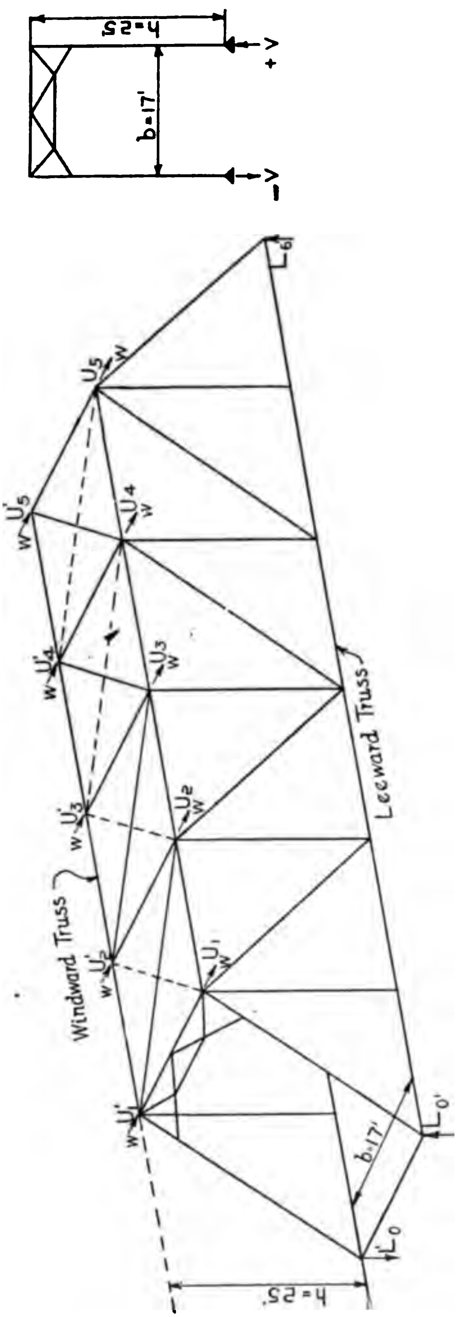


Fig. 98. Diagram Illustrating Overturning Effect of Wind on Truss.

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tical loads, each equal to V and acting downward, were placed at the hips U_1 and U_5 (see Fig. 98). The effect on the windward truss is the same as if two vertical loads, each equal to V and acting upward, were placed at the hips U_1' and U_5'.

The stresses in the leeward truss will now be worked out. The stresses in the windward truss are the same, but with opposite signs. The truss is that of Article 47. Here $V = \frac{10 \times 1.5}{2} \times \frac{25}{17} = 11\,000.$ Fig. 99 shows the truss with the loads in the correct position, the reactions each being 11.00. $V_1 = +11.00$, and $V_2 = +11.00 - 11.00 = 0$. The shears in the 2d, 3d, 4th, and 5th panels are also zero. As the shear in these panels is zero, the stress in the diagonals and vertical posts is zero $\times$ secant ϕ = zero. The stress in the hip verticals U_1L_1 and U_5L_5 is zero, as there are no loads at L_1 and L_5. The stress in the end-post is $-11.00 \times 1.28 = -14.08$. Taking the center of moments at U_1, the stress equation of $L_0L_1 = L_1L_2$ is $-L_1L_2 \times 25 + 11.00 \times 20 = 0$; whence $L_1L_2 = +8.8$. The stress in all the lower chord members will be found to be $+ 8.8$. By summing the horizontal forces at the section $a - a$, noting that, as U_1L_2 is zero, its component is also zero, there results: $+L_1L_2 + U_1U_2 = 0$; whence $U_1U_2 = -L_1L_2 = -(+8.80) = -8.80$. This is also the stress in all members of the top chord.

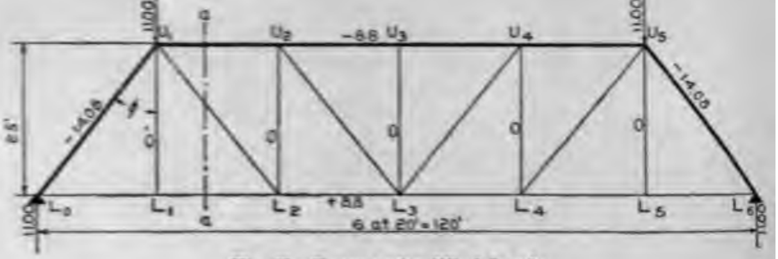


Fig. 99. Truss under Wind Loads.

It is now seen that the overturning effect of the wind on the truss causes stresses only in the end-posts and chords. The wind on the lower chord causes no overturning effect, as it is transferred directly to the abutments.

53. Overturning Effect of Wind on Train. The wind blowing upon the train tends to overturn it, and in so doing the pressure on the leeward stringer is increased and that on the windward stringer decreased by the same amount. This difference in pressures is transferred to the floor-beam and then to the panel points (see Fig. 100), where its value is:

$$\pm L = \frac{W \times (8.5 + a)}{b},$$

where W = Panel load due to wind on train;

8.5 = A constant established by the Specifications (see Article 26, p. 15);

a = Distance from base of rail to center line of lower chord. It may be taken as 3 feet in most cases, as this is approximately the usual depth of floor).

b = Distance center to center of trusses.

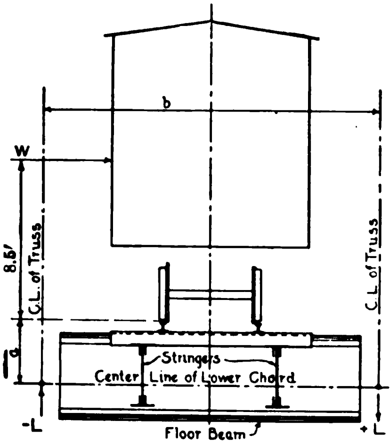


Fig. 100. Illustrating Overturning Effect of Wind on Train.

For the case in hand, $W = 20 \times 450 = 9\,000$. Therefore,

$$L = \pm \frac{9\,000 \times (8.5 + 3)}{17} = \pm 6\,090 \text{ pounds.}$$

The action of the wind in tending to overturn the train is the same as if the truss were under a live panel loading of L, the panel load L acting upward on the windward and downward on the leeward truss.

The chord stresses due to this will be proportional to the dead-load stresses in the same ratio as this panel load L is to the dead panel load. For the truss of Article 47, this ratio is $\frac{6\,090}{20\,000} = 0.303$, and the chord stresses caused by the overturning effect of the wind on the train (see Table VII) are:

$$U_1U_2 = -64.0 \times 0.303 = -19.39$$
$$U_2U_3 = -72.0 \times 0.303 = -21.82$$
$$L_0L_1 = L_1L_2 = +40.0 \times 0.303 = +12.12$$
$$L_2L_3 = +64.0 \times 0.303 = +19.39$$

The stress in U_1L_1 is +6.09, the panel load at L_1.

The maximum positive shears are:

$$V_1 = \frac{6.09}{6}(1 + 2 + 3 + 4 + 5) = +15.22$$

$$V_2 = \frac{6.09}{6}(1 + 2 + 3 + 4) = +10.15$$

$$V_3 = \frac{6.09}{6}(1 + 2 + 3) = +6.09$$

$$V_4 = \frac{6.09}{6}(1 + 2) = +3.05$$

and the maximum web stresses are found to be:

$$\begin{aligned} L_0U_1 &= -15.22 \times 1.28 = -19.50 \\ U_1L_2 &= +10.15 \times 1.28 = +13.00 \\ U_2L_3 &= +\ 6.09 \times 1.28 = +\ 7.80 \\ U_3L_4 &= +\ 3.05 \times 1.28 = +\ 3.91 \end{aligned}$$

$$\begin{aligned} U_2L_2 &= -6.09 \\ U_3L_3 &= -3.05 \end{aligned}$$

It is unnecessary to compute the shears further than one panel past the middle of the span, as only the maximum stresses are usually required.

The wind stresses from various causes are grouped together and given in Table X.

From Table X it is seen that large wind stresses occur in some of the members. Most specifications require that the stresses due to wind shall be neglected in the design unless they exceed 25 per cent of the sum of the dead-load and live-load stresses.

The subject of wind stresses does not ordinarily receive the consideration it should have; in fact, it appears to be common practice, in the case of spans up to 200 feet, to neglect the action of the wind in all members of the bridge except the top and bottom lateral diagonals, the top struts, the portal, and the bending effect in the end-post. For the last two effects mentioned, see the next succeeding article.

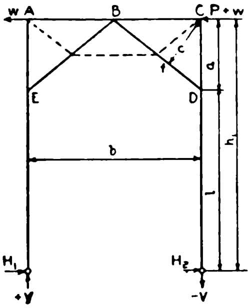


Fig. 101. Style of Portal Bracing in Common Use on Spans up to 250 Feet.

54. **Portals and Sway Bracing.** One-half of the wind on the top chord is transferred to the hips $U_1'U_1$ and $U_6'U_6$. From there it is carried to the abutments by means of the portal bracing and the end-posts. Various styles of portal bracing are in use, but few are so easily analyzed and constructed as that of Fig. 101. This form

TABLE X

Wind Stresses in Pratt Truss

WEB MEMBERS

OVERTURNING	L_0U_1	U_1L_2	U_2L_3	U_3L_4	U_1L_1	U_2L_2	U_3L_3
Wind Right							
on Truss	−14.08	0	0	0	0	0	0
on Train	−19.50	−13.00	+7.80	+3.91	+6.09	−6.09	−3.05
Wind Left							
on Truss	+14.08	0	0	0	0	0	0
on Train	+19.50	−13.00	−7.80	−3.91	−6.09	+6.09	+3.05
Maximum + Stress	+33.58	+13.00	+7.80	+3.91	+6.09	+6.09	+3.05
Maximum − Stress	−33.58	−13.00	−7.80	−3.91	−6.09	−6.09	−3.05

CHORDS

MEMBER		L_0L_1	L_1L_2	L_2L_3	U_1U_2	U_2U_3
Direct						
	Wind Right	0 0	+ 8.82 +26.46	+14.12 +42.36	0	+ 5.29
	Wind Left	− 8.82 −26.46	−14.12 −42.36	−15.88 −47.64	− 5.29	− 7.06
Overturning Truss						
	Wind Right	+ 8.80	+ 8.80	+ 8.80	− 8.80	− 8.80
	Wind Left	− 8.80	− 8.80	− 8.80	+ 8.80	+ 8.80
Overturning Train						
	Wind Right	+12.12	+12.12	+19.39	−19.39	−21.82
	Wind Left	−12.12	−12.12	−19.39	+19.39	+21.82
Maximum + Stress		+20.92	+56.20	+84.67	+28.19	+35.91
Maximum − Stress		−56.20	−77.40	−91.71	−33.48	−37.68

LATERAL SYSTEMS

MEMBER	$U_1'U_2$	$U_2'U_3$	$U_2'U_2$	$U_3'U_3$	$L_0'L_1$	$L_1'L_2$	$L_2'L_3$
Wind Right							
on Truss	+6.95	+2.32	−3.0	−1.5	+11.60	+ 6.95	+ 2.32
on Train					+34.75	+23.15	+13.91
Wind Left							
on Truss	0	0	−3.0	−1.5	0	0	0
on Train					0	0	0
Maximum	+6.95	+2.32	−3.0	−1.5	+46.35	+30.10	+16.23

The stresses in $L_1'L_1$, $L_2'L_2$, and $L_3'L_3$ are not given in the above table. These members are the floor-beams, and the small stress due to wind is neglected in their design.

is at present being used almost universally on all spans up to 200 feet.

Let it be required to analyze a portal of this form, all the distances being as indicated in Fig. 101; and let:

w = Wind panel load of upper chord;
m' = Number of panels in upper chord;

Then,

$$P = (m' - 1)\,w;$$

and,

$$V = \pm\left\{(P + w) + w\right\}\frac{h_1}{b};$$

also,

$$H_1 = H_2 = \left\{(P + w) + w\right\} \div 2.$$

The stress in BC, the center of moments being at D, is:

$$S_{BC} = -\frac{(P + w)\,a + H_2 l}{a} = -\left\{(P + w) + H_2\frac{l}{a}\right\}.$$

The stress in AB, the center of moments being at E, is:

$$S_{AB} = +\frac{wa + H_1 l}{a} = +w + H_1\frac{l}{a}.$$

For the stress in BD, the center of moments is taken at C, and the perpendicular distance c to BD is determined. The stress in BD, then, is:

$$S_{BD} = +H_2\frac{h_1}{c}.$$

The stress in BE is:

$$S_{BE} = -H_1\frac{h_1}{c}.$$

It must be remembered that h_1 is not the height of the truss, but is the length of the end-post from L_0 to U_1.

For the truss of Article 47, $w = 1.5$; $m' = 4$; and $P = 4.5$. The value $h_1 = (20^2 + 25^2)^{\frac{1}{2}} = 32.0$ feet. The distance a must be so chosen that BD will not interfere with engines or other traffic which passes through the bridge. It will be assumed as 5 feet in this case. Then $V = (4.5 + 1.5 + 1.5)\,\frac{32}{17} = 14.08$; and $H_1 = H_2 = \frac{7.5}{2} = 3.75$; whence,

$$S_{BC} = -\left(6.0 + 3.75 \times \frac{27}{5}\right) = -26.25$$

$$S_{AB} = 1.5 + 3.75 \times \frac{27}{5} = +21.75$$

The distance $BD = \sqrt{\overline{BC}^2 + \overline{CD}^2} = \sqrt{8.5^2 + 5.0^2} = 9.85$. Then, from similar triangles DCB and DFC, is obtained the proportion:

$$\frac{CF}{CD} = \frac{BC}{BD};$$

$$C = \frac{5 \times 8.5}{9.85} = 4.3 \text{ feet; and}$$

$$S_{BD} = +3.75 \times \frac{32.0}{4.3} = +27.90$$

$$S_{BE} = -3.75 \times \frac{32.0}{4.3} = -27.90$$

When the wind blows from the other side, the stresses in the diagonals are reversed, and those in the top are transposed. The members shown by broken lines take no stress. When the wind blows, the end-posts tend to bend as shown in Fig. 102. This is withstood by the cross-section of the post at the points E and D. The bending moment caused at these points by the wind is $H_1 \times l$ and $H_2 \times l$. For the truss under consideration,

$$M_D = M_E = 3.75 \times 27 \times 12 = 1\,215\,000 \text{ lb.-ins.}$$

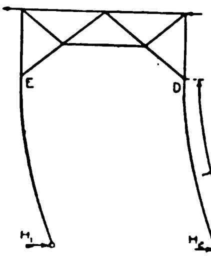


Fig. 102. Illustrating Tendency of End-Posts to Bend under Wind Load.

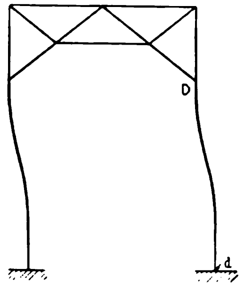


Fig. 103. Bending Tendency when End-Posts are Fixed at Lower End.

If the posts are fixed at the lower end, then they will tend to bend as shown in Fig. 103, the post resisting the bending at two points D and d. The section at each point withstands in this case only half of the moment just computed, or 1 215 000 ÷ 2 = 607 500 lb.-ins. A further discussion of this will be given in Part II, on "Bridge Design."

Various forms of sway bracing are used to connect the intermediate posts and thus stiffen the cross-section of the bridge at those points. The form of portal just given is often used, as is also the form shown in Fig. 104. Here h is the height of the truss. The braces BD are called *knee-braces*. Here w is the wind panel load of the top chord, and

$$V' = \pm\frac{2wh}{b}$$

$$H_1 = H_2 = \frac{2w}{2}$$

$$S_{BC} = -(wa + H_2l) \div a$$

$$= -(w + H_2\frac{l}{a})$$

$$S_{B'C'} = +(w + H_1\frac{l}{a})$$

$$S_{BD} = +H_2\frac{h}{c}$$

$$S_{B'D'} = -H_1\frac{h}{c}$$

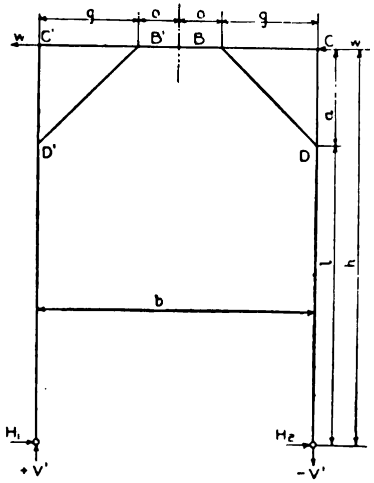


Fig. 104. A Type of Portal and Sway Bracing in Frequent Use.

The stress in $B'B$ is the direct compression due to wind right or left, and differs in accordance with the position of the top strut. There is also a bending moment at B' and B, which is:

$$M_{B'} = M_B = -V'g + H_2h.$$

The bending moment at D and D' is equal to H_2l or $H_2l \div 2$, according to whether or not the lower ends of the posts are fixed.

The determination of the stresses for the truss of Article 47 is left to the student.

When the wind is from the other side of the truss, the signs of the stresses in the knee-braces and the members $C'B'$ and CB are reversed.

55. **Final Stresses.** The class of stresses which go to make up the maximum or minimum for which the member is designed, is determined by the specifications used. The dead-load and live-load stresses are always included, and then those due to impact and wind should be added if required. In computing the maximum stresses, the *algebraic* sum should always be used. In a large majority of cases, all stresses which go to make up the maximum have the

same sign, but some exceptions have been noted, as in the middle vertical of a Pratt or Howe truss. The minimum stresses are, with rare exceptions, obtained by combining stresses with signs of opposite character.

GIRDER SPANS

56. **Moments and Shears in Floor-Beams.** In any bridge the floor-beam acts as a support for either the joists or stringers, and the moments and shears occurring in it are due to the loads which come on the joists or stringers. In a highway bridge the joists are spaced so closely that the load which they transmit to the floor-beams may be considered as uniformly distributed, providing the live load is a uniform load, in which case,

$$M = \frac{(2P_L + P_D) \times \text{panel length in inches}}{8}$$

$$V = (2P_L + P_D) \div 2,$$

where M = Maximum moment in pound-inches;
V = Maximum shear;
P_L = Live panel load;
P_D = Weight of stringers and floor material in one panel.

It will be seen that these formulæ are those for the maximum moment and shear in a uniformly loaded beam, the total load being $2P_L + P_D$.

As an example, let it be required to determine the maximum moment and shear in the floor-beam of a highway bridge whose panels are 20 feet long, and trusses 16 feet center to center, the live load being 100 pounds per square foot of floor surface, the flooring weighing 10 pounds per square foot, and there being 5 lines of joists weighing 15 pounds per linear foot, and 2 lines of joists weighing 8 pounds per linear foot.

$$P_L = \frac{16}{2} \times 20 \times 100 = 16\,000 \text{ pounds.}$$

$$P_D = 5 \times 20 \times 15 + 2 \times 20 \times 8 + 16 \times 20 \times 10 = 5\,020 \text{ pounds.}$$

Therefore,

$$M = \frac{(2 \times 16\,000 + 5\,020)\,20 \times 12}{8}$$

$$= 1\,110\,600 \text{ pound-inches at center of floor-beam.}$$

$$V = (2 \times 16\,000 + 5\,020) \div 2$$

$$= 18\,510 \text{ pounds at ends of floor-beam.}$$

In the case of a single-track railroad bridge, there are only two stringers upon which the weight of the track, the engine, and the train is supported. These join the floor-beam at points equally distant from the center of same. The weight of the ties, rails, and fastenings is usually taken at 400 pounds per linear foot of *one track*. As regards the live load, the proposition reduces itself to placing the wheel loads so that the sum of the reactions of stringers in the adjacent panels will be a maximum on the floor-beam under consideration. This is discussed in Article 47, page 87 (see Fig. 94).

In determining the values of the maximum moment and shear in the floor-beam, the case is that of a beam symmetrically loaded with two equal concentrated loads. Each load is equal to the dead weight of one stringer, one-half the track weight in one panel, and the maximum sum total of the reactions due to the wheel loads on the stringers in adjacent panels which meet at that point. This latter quantity is called the *floor-beam reaction*. For a general arrangement of the loads, see Fig. 105. The distance *a* has become standard for single-track spans, and is 6 feet 6 inches.

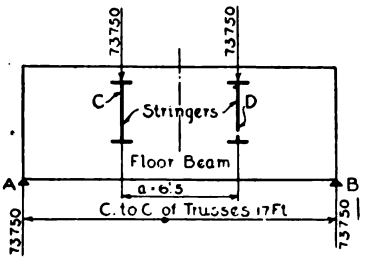


Fig. 105. Arrangement of Loads for Calculating Moments and Shears in Floor-Beams.

Let it be required to determine the maximum shears and moments in the floor-beam of the truss of Article 47.

The weight of the stringer may be obtained by the formula of Table II, and is:

$$\text{Stringer} = 20\,(123.5 + 10 \times 20) \div 2 = 3\,200 \text{ pounds.}$$

The weight of one-half of the ties, rails, etc., in one panel is:

$$\tfrac{1}{2}\ \text{Track} = (400 \times 20) \div 2 = 4\,000 \text{ pounds.}$$

The weight that comes from the engine wheels is given in Article 47, page 87 (see Fig. 94), and is 65.55. Each load is therefore the sum of all the above weights, as follows:

$$3\,200 + 4\,000 + 65\,550 = 73\,750.$$

The maximum shear (see Fig. 105) is seen to be 73 750 pounds; and the maximum moment occurs at *C* and *D*, and is:

$$M = 73\,750 \times \left(\frac{17}{2} - \frac{6.5}{2}\right) \times 12 = 4\,646\,250 \text{ pound-inches.}$$

For any particular engine the floor-beam reactions for different length panels are easily tabulated for future reference. Table XI gives the floor-beam reactions for panel lengths from 10 to 24 feet inclusive.

TABLE XI

Floor-Beam Reactions

E 40 Loading

Panel Length	Maximum Floor-Bm. Reaction	Panel Length	Maximum Floor-Bm. Reaction	Panel Length	Maximum Floor-Bm. Reaction
10	41 000	15	55 000	20	65.55
11	43 800	16	57 000	21	67.10
12	46 600	17	59 000	22	69.20
13	44 400	18	61 000	23	71.30
14	52 200	19	63 000	24	73 40

In many cases it is desirable to keep the dead-load shears and moments separate from those of the live load; and this can easily be done.

In neither of the above cases has the weight of the beam itself been taken into account. This should be done in the final design. The method of procedure is to compute the moment and shears as above; then make a provisional design of the beam. Next, compute the weight of the beam thus designed, and add the moments and shears caused by this weight to the other dead-load moments and shears; then re-design the beam and compute its weight. If this last weight varies 10 per cent from the previous weight, another re-design should be made. The above proceeding belongs to Bridge Design, Part II, and will there be treated.

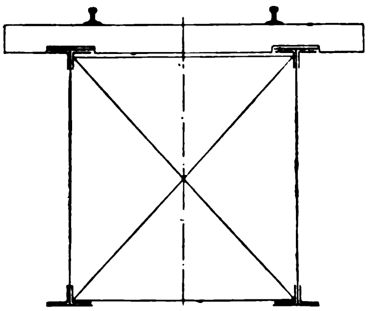


Fig. 106. Cross-Section of Deck Plate-Girder Railway Bridge.

57. **Moments in Plate-Girders.** Plate girders are of two classes —namely (1) those which have the ties or floor laid directly upon the upper flanges of the girders; these are called *deck plate-girder bridges*;

and (2) those in which the webs of the girders are connected with each other at intervals by floor-beams which in turn carry stringers or joists in exactly the same manner as in the floor system of a railroad or highway truss-bridge; this latter type is called a *through plate-girder bridge.* Figs. 106 and 107 show cross-sections of deck and through plate-girder bridges respectively, for railway service. Fig. 108 is a side view of a deck plate-girder bridge. Fig. 109 is a longitudinal section of a through plate-girder railroad bridge. The section is taken down the middle of the track. The bridge shown has 5 panels. An odd number of panels should be chosen, as this does not bring a floor-beam at the center of the span, and hence the great moment which would then be caused is avoided.

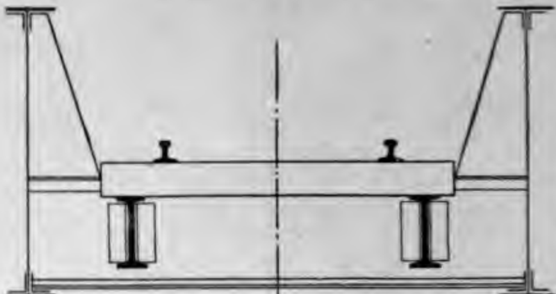


Fig. 107. Cross-Section of Through Plate-Girder Railway Bridge.

The analysis of the shears and moments of a through plate-girder is precisely the same as that for a truss bridge. The shear is constant between any two panel points as 0-1 or 1-2, etc., and the moments are computed for the points 1, 2, 3, and 4.

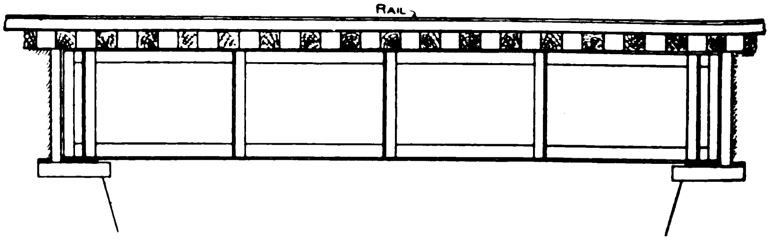


Fig. 108. Side View of Deck Plate-Girder Railway Bridge.

If wheel loads are used for moments, the relation that $K = \frac{Wn}{m} - L$ must be $+$, and that $k = \frac{Wn}{m} - (L + P)$ must be $-$, holds true when the loads are in correct position for maximum moments. Here m = the number of panels, and n = the panel under consideration and is to be reckoned from the left end; in fact, all terms have the same value as mentioned in Article 46. A careful review of Articles 44 and 46 should enable the student to follow the example which will now be given.

Example. It is required to determine the moments at the points of floor-beam support for a 5-panel through plate-girder of 75-foot span. The live loading is Cooper's E 40.

Dead-Load Moments. Through plate-girders, on account of the heavy floor system and the fact that the floor system transfers its own weight and that of the live load to the girders as concentrated loads, are about 40 per cent heavier than deck plate-girder bridges of the same span. The weight of the entire span, therefore, is;

$$1.4 \times 75\,(123.5 + 10 \times 75) = 91\ 700 \text{ pounds.}$$

Part of this 91 700 pounds (the weight of the girders themselves) acts as a uniform load; the remainder (the weight of the floor-beams and stringers) acts as concentrated loads at the points where the floor-beams join the web. Experience has shown that the weight of the floor for a single-track railroad system is about 400 pounds

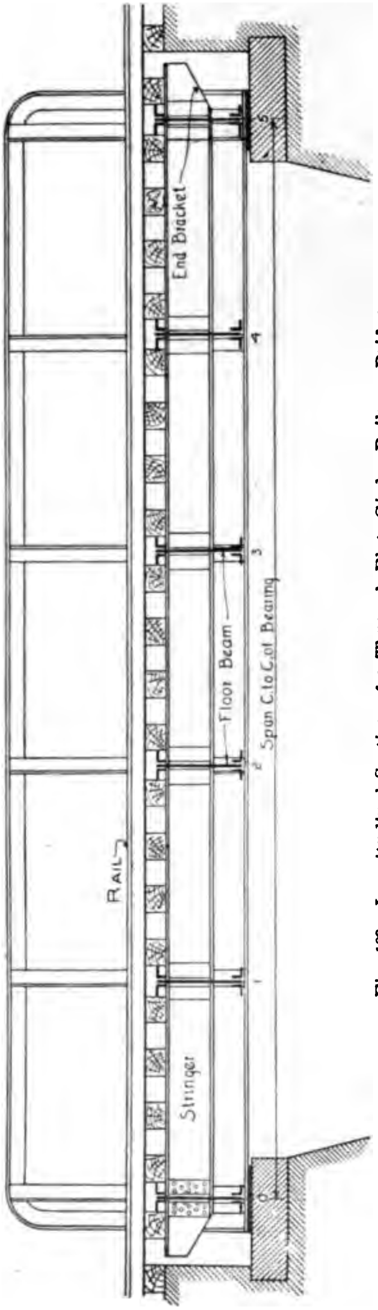


Fig. 109. Longitudinal Section of a Through Plate-Girder Railway Bridge.

TABLE XII

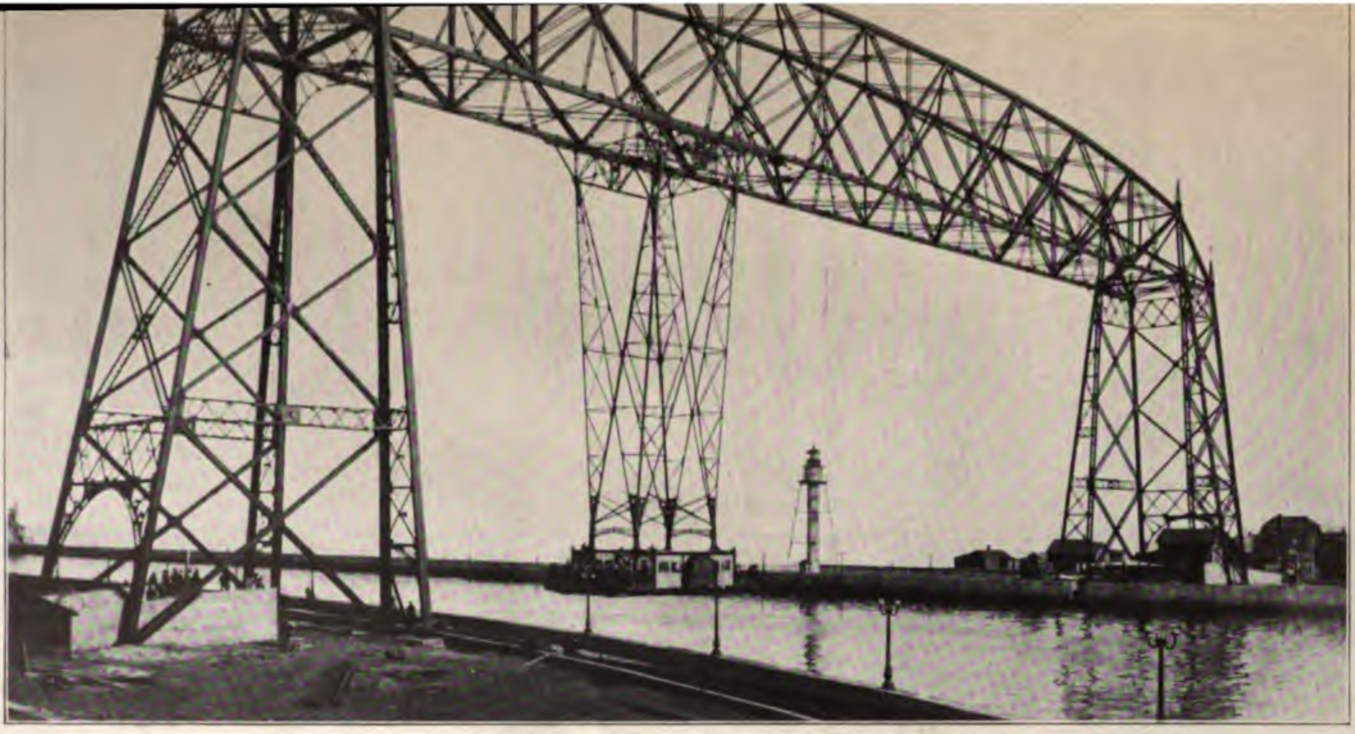
Wheel Position, Moments in a Through Plate-Girder

Panel Point	Wheel at Point	L	$\frac{Wx}{m}$	P	$L+P$	K	k	Remarks
1	2	10	$\frac{172}{5} = 34.4$	20	30	+	+	
1	3	30	$\frac{192}{5} = 38.4$	20	50	+	–	Maximum
1	4	40	$\frac{202}{5} = 40.4$	20	60	+	–	Maximum
1	5	40	$\frac{202}{5} = 40.4$	20	60	+	–	Maximum
1	6	40	$\frac{195}{5} = 39.0$	13	53	–	–	
2	4	50	$\frac{152 \times 2}{5} = 60.8$	20	70	+	–	Maximum
2	5	70	$\frac{172 \times 2}{5} = 68.8$	20	90	–	–	
3	6	90	$\frac{152 \times 3}{5} = 91.2$	13	103	+	–	Maximum
3	7	103	$\frac{172 \times 3}{5} = 103.2$	13	116	+	–	Maximum
3	8	116	$\frac{192 \times 3}{5} = 115.2$	13	129	–	–	
4	6	90	$\frac{129 \times 4}{5} = 103.2$	13	103	+	–	Maximum
4	7	103	$\frac{142 \times 4}{5} = 113.6$	13	116	+	–	Maximum
4	8	116	$\frac{152 \times 4}{5} = 121.6$	13	142	+	–	Maximum
4	9	129	$\frac{152 \times 4}{5} = 121.6$	13	142	–	–	

per linear foot. The weight of the stringers and floor-beams for this bridge is therefore 75 × 400 = 30 000 pounds, and 91 700 − 30 000 = 61 700 pounds acts as a uniform load. This 61 700 pounds is distributed over two girders, and so gives 61 700 ÷ (2×75) = say, 412 pounds per linear foot of one girder.

The dead load which is concentrated at each panel point is that due to the weight of the steel floor and the weight of ties, rails, and fastenings. It is, for one girder,

$$15 \times (400 + 400) \div 2 = 6\,000 \text{ pounds.}$$



AËRIAL BRIDGE OVER SHIP CANAL AT DULUTH, MINNESOTA

Built in 1904-05. View showing suspended car in transit. Structure rests on concrete piers, and consists of a riveted truss on riveted steel towers, the ferry car being supported by an inverted steel tower. Clear height above water, 135 ft.; clear span, 393.75 ft. Car has space for a street-car, two wagons, and two enclosed cabins; will carry 125,000 lbs.; passage made in a little over 1 minute. Motive power is electricity supplied from two separate sources, with hand-power available in case of failure of the electric current. Operates perfectly in high winds. Can handle over 33,000 passengers daily. Cost of structure, about $8,000.

The dead-load moments are now computed by the methods of Strength of Materials, and are found to be:

$M_0 = M_5 = 0$;
$M_1 = M_4 = +4\,390\,000$ pound-inches;
$M_2 = M_3 = +6\,580\,000$ pound-inches.

Live-Load Moments. The positions of the wheels for maximum moments are now determined (see Table XII).

The computations for the reactions are best arranged in tabular form. Table XIII gives the values.

TABLE XIII
Reactions for a Through Plate-Girder

Point	Wheel at Point	Equation for Reaction	Reaction
1	3	$R = (6\,708 + 192 \times 4) \div 75$	99.7
1	4	$R = (7\,668 + 212 \times 4 - 10 \times 78) \div 75$	103.2
1	5	$R = (8\,728 + 232 \times 4 - 10 \times 83 - 20 \times 75) \div 75$	97.8
2	4	$R = (4\,632 + 152 \times 7) \div 75$	76.0
3	6	$R = (4\,632 + 152 \times 6) \div 75$	73.9
3	7	$R = (5\,848 + 172 \times 3) \div 75$	84.9
4	6	$R = (2\,851 + 129 \times 4) \div 75$	44.9
4	7	$R = (3\,496 + 142 \times 4) \div 75$	54.2
4	8	$R = (4\,632 + 152 \times 2) \div 75$	65.4

The live-load moments are computed as follows:

Point 1
Wheel 3, $M = 99.7 \times 15 - 230 = 1\,265\,000$ pound-feet.
Wheel 4, $M = 103.2 \times 15 - 20 \times 5 - 20 \times 10 = 1\,247\,000$ pound-feet.
Wheel 5, $M = 97.8 \times 15 - 20 \times 5 - 20 \times 10 = 1\,167\,000$ pound-feet.

Point 2 Wheel 4, $M = 76.0 \times 30 - 480 = 1\,800\,000$ pound-feet.

Point 3
Wheel 6, $M = 73.9 \times 45 - 1\,640 = 1\,785\,000$ pound-feet.
Wheel 7, $M = 84.9 \times 45 - 2\,155 = 1\,665\,000$ pound-feet.

Point 4
Wheel 6, $M = 44.9 \times 60 - 1\,640 = 1\,054\,000$ pound-feet.
Wheel 7, $M = 54.2 \times 60 - 2\,155 = 1\,097\,000$ pound-feet.
Wheel 8, $M = 65.4 \times 60 - 2\,851 = 1\,073\,000$ pound-feet.

The above values show that the greatest live-load moments are:

M at Point 1, by wheel 3 = 1 265 000 pound-feet
M at " 2, " " 4 = 1 800 000 " "
M at " 3, " " 4 = 1 800 000 " "
M at " 4, " " 3 = 1 265 000 " "

The last two values are obtained when the load comes on the bridge from the left. Inspection of the results obtained at points 3 and 4 when the load comes on from the right, shows that they are considerably smaller than the results obtained at their symmetrical points 1 and 2, and therefore it was not necessary to determine the moments for any points to the right of the center. This is true of all girder spans, deck or through.

The method of procedure when the girder is a deck plate-girder is the same as that just illustrated, except that in the computation of the dead-load moments there is no concentration of certain portions of the dead load, the weight of the girders themselves being a uniform load, as is also the weight of the ties and rails or, if it be a highway bridge, the floor-joists which run transversely. Highway spans are seldom built of deck plate-girders, it being preferable to use the through girders, as then the girders themselves serve as a railing and keep the traffic confined to the roadway. The girder span is usually divided into ten equal divisions, the points of division being called the *tenth-points*. The shears and moments are computed for the center point and those points which lie to the left of the center. After the values are computed, they are laid off as ordinates, with the corresponding tenth-points as abscissæ. A curve is then drawn through their upper ends, and the *curve of maximum shears or moments* is the result. To get the maximum shear or moment at any point other than a tenth-point, the ordinate is scaled at the desired point.

EXAMPLE. Let it be required to determine the maximum moments at the tenth-points of a 100-foot-span deck plate-girder.

Dead-Load Moments. The weight of steel in the span is 100 (123.5 + 10 × 100) = 112 350 pounds, and the weight of the track is 400 × 100 = 40 000 pounds, making a total of 152 350 pounds, or 152 350 ÷ (2 × 100) = say, 762 pounds per linear foot per girder. The dead-load moments are now determined according to the methods of Strength of Materials, and are:

M_1 = 342 800 pound-feet
M_2 = 609 400 " "
M_3 = 799 840 " "
M_4 = 914 100 " "
M_5 = 952 200 " "

Live-Load Moments. The determination of the wheel load positions is made by the use of the formulæ $K = (\frac{W'n}{m} - L)$ and $k = \frac{W'n}{m} - (L + P)$; only, in this case, n is the number of divisions from the left support to the section, and m is the number of divisions into which the girder is divided.

The determination of the wheel positions is given in Table XIV.

TABLE XIV

Wheel Positions, Moments in Deck Plate-Girder

Point	Wheel at Point	L	$\frac{W'n}{m}$	P	$L+P$	K	k	Remarks
1	2	10	258 × 0.1 = 25.8	20	30	+	−	Maximum
1	3	20	261 × 0.1 = 26.1	20	40	+	−	"
1	4	20	254 × 0.1 = 25.4	20	40	+	−	"
1	5	20	242 × 0.1 = 24.2	20	40	+	−	"
1	6	20	240 × 0.1 = 24.0	13	33	+	−	"
1	7	13	230 × 0.1 = 23.0	13	26	+	−	"
2	2	10	232 × 0.2 = 46.4	30	30	+	+	
2	3	30	245 × 0.2 = 49.0	20	50	+	−	Maximum
2	4	50	258 × 0.2 = 51.6	20	70	+	−	"
2	5	60	261 × 0.2 = 52.2	20	80	−	−	
3	3	30	232 × 0.3 = 69.6	20	50	+	+	
3	4	50	232 × 0.3 = 69.6	20	70	+	−	Maximum
3	5	70	245 × 0.3 = 73.5	20	90	+	−	"
3	6	80	261 × 0.3 = 78.3	13	93	−	−	
4	4	50	212 × 0.4 = 84.8	20	70	+	+	
4	5	70	232 × 0.4 = 92.8	20	90	+	−	Maximum
4	6	90	245 × 0.4 = 98.0	13	103	+	−	"
4	7	103	258 × 0.4 = 103.2	13	116	+	−	"
4	8	106	261 × 0.4 = 104.4	13	119	−	−	
5	6	90	232 × 0.5 = 116.0	13	103	+	+	
5	7	103	232 × 0.5 = 116.0	13	116	+	0	Maximum
5	8	116	245 × 0.5 = 127.5	13	129	+	−	"
5	9	129	258 × 0.5 = 129.0	13	142	0	−	"
5	10	132	274 × 0.5 = 137.0	10	142	+	−	"
5	11	102	244 × 0.5 = 122.0	20	122	+	0	"
5	12	102	234 × 0.5 = 117.0	20	122	+	−	"
5	13	102	224 × 0.5 = 112.0	20	122	+	−	"
5	14	122	234 × 0.5 = 117.0	20	132	−	−	

While many wheels on point 1 satisfy the condition, the greatest moment will occur when one of the large drivers is at the point, and it is therefore unnecessary to examine the point for other wheels. The same is true at the center point, 5, the maximum occurring under one of the heavy driver wheels. The reactions and the computations for the same are given in Table XV.

TABLE XV

Reactions for a Deck Plate-Girder

POINT	WHEEL AT POINT	EQUATION FOR REACTION	REACTION
1	2	$R = (12\,041 + 5 \times 258) \div 100$	133.31
1	3	$R = (12\,599 + 4 \times 261) \div 100$	136.43
1	4	$R = (11\,984 + 4 \times 254) \div 100$	130.00
1	5	$R = (11\,334 + 4 \times 234 \div 2 \times 4^2 \div 2) \div 100$	122.86
2	3	$R = 12\,041 \div 100$	120.41
2	4	$R = (12\,041 + 5 \times 258) \div 100$	133.31
3	4	$R = 10\,816 \div 100$	108.16
3	5	$R = 12\,041 \div 100$	120.41
4	5	$R = (8\,728 + 4 \times 232) \div 100$	96.56
4	6	$R = (10\,816 + 4 \times 245) \div 100$	117.96
4	7	$R = (12\,041 + 4 \times 258) \div 100$	130.73
5	7	$R = (8\,728 + 8 \times 232) \div 100$	105.84
5	8	$R = 12\,041 \div 100$	120.41
5	9	$R = (12\,041 + 5 \times 258) \div 100$	133.31
5	10	$R = (13\,904 + 2 \times 274) \div 100$	144.52
5	11	$R = (11\,334 + 5 \times 234 + 2 \times 5^2 \div 2) \div 100$	125.29
5	12	$R = (9\,514 + 10 \times 214 + 2 \times 10^2 \div 2) \div 100$	117.54
5	13	$R = (7\,794 + 15 \times 194 + 2 \times 15^2 \div 2) \div 100$	109.29

TABLE XVI

Maximum Moments in a Deck Plate-Girder

POINT	WHEEL AT POINT	EQUATION FOR MOMENT	MOMENT IN POUND-FEET
1	2	$M = 133.31 \times 10 - 80$	1 253 100
1	3	$M = 136.43 \times 10 - 5 \times 20$	1 264 000
1	4	$M = 130.00 \times 10 - 5 \times 20$	1 200 000
1	5	$M = 122.86 \times 10 - 5 \times 20$	1 128 000
2	3	$M = 120.41 \times 20 - 230$	2 178 200
2	4	$M = 133.31 \times 20 - 480$	2 186 200
3	4	$M = 108.16 \times 30 - 480$	2 764 800
3	5	$M = 120.41 \times 30 - 830$	2 782 300
4	5	$M = 96.56 \times 40 - 830$	3 032 400
4	6	$M = 117.96 \times 40 - 11\,640$	3 078 400
4	7	$M = 130.73 \times 40 - 2\,155$	3 074 200
5	7	$M = 105.84 \times 50 - 2\,155$	3 137 000
5	8	$M = 120.41 \times 50 - 2\,851$	3 169 500
5	9	$M = 133.31 \times 50 - 3\,496$	3 169 500
5	10	$M = 144.52 \times 50 - 4\,072$	3 154 000
5	11	$M = 125.29 \times 50 - 3\,068$	3 196 500
5	12	$M = 117.54 \times 50 - 2\,658$	3 219 000
5	13	$M = 109.29 \times 50 - 2\,248$	3 216 500

Table XVI gives the computations of the live-load moments at the tenth-points, the final results being in pound-feet.

Whenever any loads were off the left end of the bridge, the lines 7 to 16 of the engine diagram were used (Fig. 85). For example, with wheel 10 at 5, wheel 1 would be off the left end. By looking in the second space of line 8, there is found the quantity 13 904, which is the moment of wheels 2 to 18 inclusive about a point directly under wheel 18. Just to the right of the vertical line through wheel 18, is the value 284, which is the weight of wheels 1 to 18 inclusive; but this must be decreased by 10, the weight of wheel 1, as that wheel is off the span. As wheel 18 is 2 feet from the right end of the girder, the moment about the point is 13 904 + 274 × 2. By looking in the second space of line 16, the value 4 072 is found. This is the value of the moment of loads 2 to 9 inclusive about a point directly under wheel 10, and must be subtracted from the moment of the reaction in order to get the moment at 5 for this loading. See Articles 21 and 47 for further information regarding the use of the values in lines 7 to 16 of the engine diagram.

By the help of differential calculus it can be proved that *the greatest possible moment does not occur at the middle of a beam loaded either with concentrated loads or with concentrated loads followed by a uniform load, but it occurs under the load nearest the middle of the beam when the loads are so placed that the middle of the beam is half way between the center of gravity of all the loads and the nearest load.*

The wheel which produces this greatest moment is the same one which produces the maximum moment at the middle of the beam. The exact solution of this problem involves the use of quadratic equations, but for all practical purposes the following rule will suffice:

> Place the loading so that the wheel which produces the maximum moment at the middle of the beam is at that point. Find the distance of the center of gravity of all the loads from the right end. Move the loads so that the middle of the beam is half way between the center of gravity as found above and the load which produced the maximum moment at the middle of the beam. Find the moment under that load, with the loads in the position just mentioned.

For the case in hand, wheel 12 at 5 gives the maximum moment. The moment at the right end of the span, wheel 12 being at 5, is:

$$9\,514 + 10 \times 214 + 2 \times \overline{10}^2 \div 2 = 11\,754\,000 \text{ pound-feet.}$$

The center of gravity is $\frac{11\ 754\ 000}{234} = 50.2$ feet from the right support, or 0.2 foot to the left of the center of the girder. Now place wheel 12 one-tenth of a foot to the right of the center, and determine the moment under it. The reaction will be:

$$R = (9\ 524 + 9.9 \times 234 + 2 \times \overline{9.9}^2 \div 2) \div 100 = 117.306;$$

and,

$$M = 117.306 \times 50.1 - 2\ 658 = 3\ 219\ 306 \text{ pound-feet.}$$

In this particular case the difference between the greatest moment possible and the greatest moment at the middle is not sufficient to warrant the extra labor involved in computing it. In general it may be said that *if the greatest moment possible occurs within six inches of the middle of the beam, it is not necessary to compute it, the moment at point 5 being taken.*

58. **Shears in Plate-Girders.** In the case of through plate-girders, the maximum live-load shears are determined by placing the wheels in such a position that $Q = \frac{W}{m} - G$ is +, and $q = \frac{W}{m} - (G + P)$ is −.

In these equations, m is the number of panels into which the span is divided; and the other quantities are the same as given in Article 45, which should now be reviewed.

For example, let it be required to determine the dead and live load shears in the through plate-girder of Article 57, p. 111.

The weight of one girder = 61 700 ÷ 2 = 30 850 lbs.
" " " ½ the floor = 5 × 6 000 = 30 000 lbs.

Total weight on one girder = 60 850 lbs.

The dead-load shears are then computed by the methods given in Strength of Materials, and are given as follows, it being remembered that the concentrated load which comes at the end is one-half a panel load, or 3 000 pounds:

$$V_0 = 60\ 850 \div 2 = 30\ 475 \text{ lbs.} = \text{end shear};$$

$$V_1 = 30\ 475 - 3\ 000 - \frac{30\ 850}{5} = 21\ 305 \text{ lbs.};$$

$$V_2 = 30\ 475 - 3\ 000 - 6\ 000 - 2 \times \frac{30\ 850}{5} = 9\ 135 \text{ lbs.}$$

$$V_3 = 30\ 475 - 3\ 000 - 2 \times 6\ 000 - 2 \times \frac{30\ 850}{5} = 3\ 135 \text{ lbs.}$$

$$V_4 = 0,$$

where V_0 = the shear at the end; V_1 = the shear just to the left of point 1; V_2 = the shear just to the left of point 2; V_3 = the shear just to the right of point 2; and V_C = the shear at the middle of the girder.

The determination of the wheel load position for maximum live-load shears is given in Table XVII. By comparing the formulæ Q and K, it will be seen that for the first panel $Q = K$, and $q = k$, as $n = 1$. The position of wheel loads for maximum moments at point 1 is the same as for maximum shear in the first panel. According to Table XII, wheels 3, 4, and 5 at point 1 all give maximum shears in the first panel. In this case, as in previous ones, only the shear for the first position of the loading found for any particular point will be determined, as the difference between this and the other cases is too small to warrant the additional labor necessary in computing them. It is evidently unnecessary to go past panel 3, as only the maximum shears are required.

TABLE XVII

Wheel Positions, Shears in a Through Plate-Girder

($m = 5$)

POINT	WHEEL AT POINT	G	$\frac{W}{m}$	P	$P+G$	Q	q	REMARKS
1								See Table XII, and text above this table
2	2	10	142 ÷ 5 = 28.4	20	30	+	–	Maximum
2	3	30	152 ÷ 5 = 30.4	20	50	+	–	Maximum
2	4	40	152 ÷ 5 = 30.4	20	60	–	–	
3	2	10	116 ÷ 5 = 23.2	20	30	+	–	Maximum
3	3	30	116 ÷ 5 = 23.2	20	50	–	–	

For wheel 3 at point 1, the left reaction (see Table XIII) is 99.7. That portion of wheels 1 and 2 which is transferred to point 0 is 230 ÷ 15 = 15.33; and the shear in the first panel, therefore, is:

$$V_1 = 99.70 - 15.33 = +84.37.$$

For wheel 2 at point 2, the left reaction is:

$$R = (3\,496 + 142 \times 5) \div 75 = 56.10;$$

therefore,

$$V_2 = 56.10 - \frac{80}{15} = +50.67.$$

A computation with wheel 3 at point 2 will give a shear only 560 pounds greater, which difference would not influence the design to any appreciable extent.

For wheel 2 at point 3, the left reaction is:

$$R = (2\,155 + 116 \times 1) \div 75 = 30.30;$$

therefore,

$$V_3 = 30.3 - \frac{80}{15} = +24.97.$$

When the girder is a deck one, the computation of the dead shears is very much simplified, as all of the load is uniform.

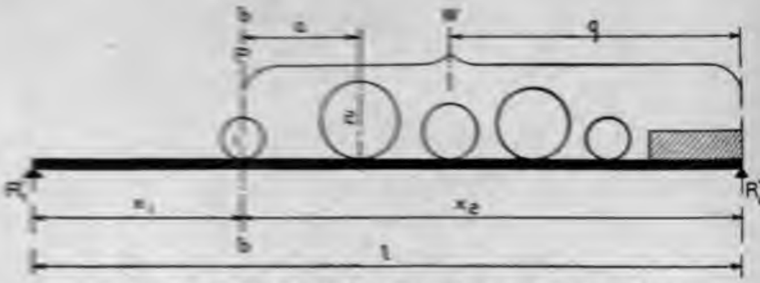


Fig. [illegible]. Beam under Wheel Loads Followed by Uniform Load.

Let it be required to determine the dead and live load shears at the tenth-points of the deck plate-girder of Article 57.

The total weight of the girder and track is 76 175 lbs.

$$V_0 = 76\,175 \div 2 = -38\,088 \text{ pounds.}$$

$$V_1 = 38\,088 - \frac{76\,175}{10} = -30\,470 \text{ pounds.}$$

$$V_2 = 38\,088 - \frac{2}{10} \times 76\,175 = -22\,450 \text{ pounds.}$$

$$V_3 = 38\,088 - \frac{3}{10} \times 76\,175 = -15\,230 \text{ pounds.}$$

$$V_4 = 38\,088 - \frac{4}{10} \times 76\,175 = -\ 7\,618 \text{ pounds.}$$

$$V_5 = 0.$$

The position of the wheel loads to produce the maximum shear cannot be determined by the same relation as that used in structures which have a system of floor-beams and stringers, for here not a portion, but all of the load to the left of the section, must be subtracted from the left reaction in order to give the shear.

The correct relation for the wheel load position will now be deduced.

Let Fig. 110 represent a beam of span l loaded with a series of wheel loads followed by a uniform load. Let P equal the weight of the first wheel, W equal the weight of all the loads, and g the distance from the center of gravity of all of the loads to the right abutment. The distance between the first and second wheel centers is a, and the first wheel is at the section $b-b$ at a distance x_1 from the left support. Then,

$$R_1' = \frac{Wg}{l};$$

and

$$V'_{b-b} = R_1' - \text{(loads to left of section)} = \frac{Wg}{l}.$$

Now, assume that the loads move forward the distance a. The wheel 2 will be at section $b-b$, and Fig. 111 will represent the position of the loads. Then,

$$R_1'' = \frac{W(g+a)}{l};$$

and

$$\begin{aligned} V''_{b-b} &= R_1'' - P \\ &= \frac{W(g+a)}{l} - P \\ &= \frac{Wg}{l} + \frac{Wa}{l} - P. \end{aligned}$$

It is now evident that in order to get the greatest shear at section $b-b$, wheel 2 must be placed at the section whenever V''_{b-b} is greater than V'_{b-b}. Then,

$$V''_{b-b} > V'_{b-b};$$

$$\frac{Wg}{l} + \frac{Wa}{l} - P > \frac{Wg}{l}.$$

Now, canceling out the term $\frac{Wg}{l}$, which appears on both sides of the equation, there results:

$$\frac{Wa}{l} > P.$$

For the engine under consideration, $a = 8$ feet, and $P = 10\,000$ pounds, and the equation reduces to:

$$W > \frac{10.0}{8}l;$$

$$W > 1\frac{1}{4}l,$$

which is to say that when the load on the girder is greater than 1¼ times the span, then wheel 2 should be placed at the section in order to give the maximum shear.

For loading E 40, the following is true:

For all sections up to and including the center of all spans, place wheel 2 at the section to give the maximum shear.

In Fig. 111 it is immaterial whether or not any additional loads come on the span at the right end when the loads move forward the distance a, as they would only tend to increase the left reaction and therefore the shear V''_{b-b}. If the relation deduced is true for the case when no extra loads come on at the right end, it will be true when they do.

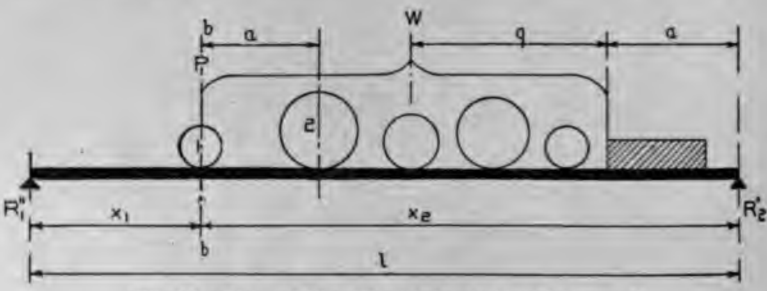


Fig. 111. Beam of Fig. 110 with Loads Moved Forward.

The live-load shears at the left end and at the tenth-points, wheel 2 being at the section in all cases, are computed from the general formula, which is:

$$V = R - \Sigma P,$$

in which,

R = Left reaction;

ΣP = All loads to left of section, and is equal to 10 000 pounds for all sections except the end of the girder.

The computations and results can be conveniently placed in tabular form, and are given in Table XVIII.

In order to illustrate the use of the relation $W > 1\frac{1}{4}l$, let point 3 in the above span be taken. Place wheel 2 at point 3; then, as wheels 1 to 13 are on the girder, the total weight W is 212. As l = 100, $1\frac{1}{4}l$ = 125. Therefore, as 212 is greater than 125, wheel 2 is the correct wheel.

TABLE XVIII

Maximum Shears in a Deck Plate-Girder

Point	Reaction Equation	R	ΣP	V	Remarks
0	$(13\,904+4\times274)\div100$	150.00	0	150.00	Wheel 18, 4 ft. from rt. end
1	$(12\,041+5\times258)\div100$	133.31	10	123.31	Wheel 16, 5 ft. from rt. end
2	$10\,816\div100$	108.16	10	98.16	Wheel 15 at right end
3	$(\ 7\,668+4\times212)\div100$	85.16	10	75.16	Wheel 13, 4 ft. from rt. end
4	$(\ 5\,848+4\times172)\div100$	65.36	10	55.36	Wheel 11, 4 ft. from rt. end
5	$(\ 4\,632+2\times152)\div100$	49.36	10	39.36	Wheel 10, 2 ft. from rt. end

The curves of maximum live-load moments and shears are shown in Fig. 112. They should always be drawn. From them the shear or moment at any desired section can be determined. For example, let it be desired to determine the maximum live-load shear and moment at a point 24 feet from the left end of the girder. By drawing the ordinate, shown by a broken line in Fig. 112, and scaling, the following values are found:

V_{24} = 88 000 pounds;
M_{24} = 2 440 000 pound-feet.

A similar set of curves for the dead-load shears and moments should be made. The set for the deck plate-girder in hand is shown in Fig. 113. These are easily constructed by laying off the maximum values of the shear at the end, and the maximum value of the moment at the center. The shear curve is a straight line from the end to the center, while the moment curve is a parabola from the center to the end.

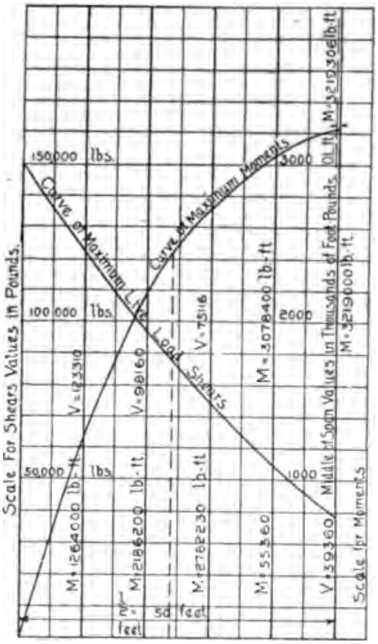


Fig. 112. Curves of Maximum Live-Load Moments and Shears.

The stresses in the lateral systems of plate-girders are computed in a manner the same as that employed for the lateral systems of trusses, the unit-load being taken according to the specifications used.

Stresses in Plate-Girders. The stresses in plate-girders ed in the Instruction Paper on Steel Construction, Part IV, to 263, and the student is referred to this treatise for information egarding this subject.

stress in the flange is seen to depend upon the distance from gravity to center of gravity. This distance, in turn, depends

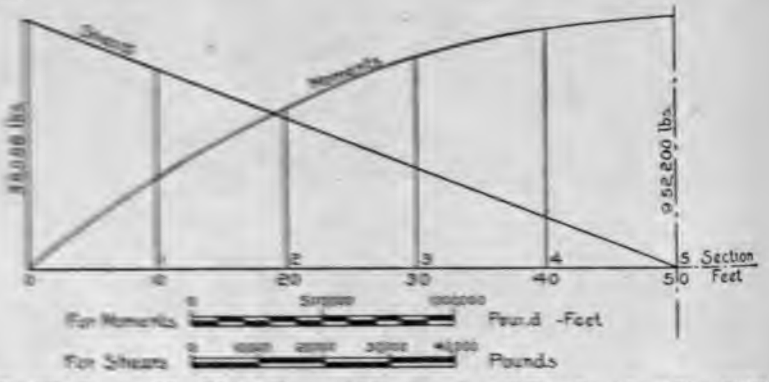


Fig. [illegible]. Curves of Dead-Load Shears and Moments in a 100-Foot Span Deck Plate-Girder.

upon the depth of the girder. Certain approximate rules have been proposed [illegible] to determine this, but the following formula will give the width of the web plate in accordance with best modern practice:

$$d = \frac{l}{0.005\,l + 0.543},$$

in which

d = Width of the web plate, in the even inch;

l = Span in feet.

For example, let it be required to determine the width of the web plate of a plate-girder of 80-foot span center to center of end bearings.

$$d = \frac{80}{0.005 \times 80 + 0.543} = \frac{80}{0.94} = 85.2 \text{ (say 86) inches.}$$

If the resultant value had been 85 inches, the width would have been taken as either 84 or 86. The reason for this is that the wide plates kept in stock at the mills are usually the even inch in width and can therefore be procured more quickly than if odd-inch widths were ordered, in which case the purchaser would be forced to wait until they were rolled—often a period of several months. The distance back to back of flange angles, the so-called *depth* of girder, is one-half inch more than the width of the web. This is due to the fact that each pair of flange angles extend one-fourth inch beyond the edge of the web plate, so as to keep any small irregularities caused on the edge of the web plate by the rolling, from extending beyond the backs of the angles.

EXERCISES AND PROBLEMS

1. Determine the maximum positive shears in the first six panels of a 9-panel 114-foot Pratt truss, the live panel load being 8.0. Use the exact and also the conventional method.

ANSWER:

	x	EXACT SHEARS	CONVENTIONAL SHEARS
V_1	16	$+32.00$	$+32.00$
V_2	14	$+24.54$	$+24.90$
V_3	12	$+18.05$	$+18.70$
V_4	10	$+12.48$	$+13.35$
V_5	8	$+\ 7.85$	$+\ 8.90$
V_6	6	$+\ 4.50$	$+\ 5.34$

2. Find the maximum and minimum stresses in L_1U_2 and U_3L_3 of an 8-panel 160-foot through Warren truss. Height 20 ft.; dead panel load 10.00, all on lower chord; live panel load 12.00.

ANSWER: In L_1U_2: d. l., -28.00; l. l., -35.30 and $+1.68$; max., -63.30; min., -26.32. In U_3L_3: d. l., $+16.80$; l.l., $+25.20$ and -5.04; max., $+42.00$; min., $+11.76$.

3. In the truss of Problem 2, determine the maximum stress in L_2L_3 by the method of moments, and also by the tangent method.

ANSWER: d. l. $= +67.50$; l. l. $= +81.00$; max. $= +148.50$.

4. Determine the dead-load stresses in the members U_2L_2 and L_4U_5 of a 9-panel 180-foot through Warren truss. Height is 24 feet; dead panel load is 10.0, one-third being at each panel point of the upper chord, and two-thirds being at each panel point of the lower chord.

ANSWER: $U_2L_2 = +30.60$; $L_4U_5 = -1.80$.

5. Determine the stress in the counter of a through Howe truss of 8 panels and 160-foot span. Height is 30 ft.; dead panel load, 9.6; live panel load, 11.5.

ANSWER: -4.59.

6. **In the truss of Problem 5, determine the maximum and minimum stress in U_2L_2, L_2U_3, and L_3U_4.**

ANSWER:

	U_2L_2	L_2U_3	L_3U_4
d. l. l. l. l. l.	+20.80 +30.30 −14.40	−17.30 −25.90 + 5.18	− 5.76 −17.30 0.00
Max. Min.	+15.10 +19.36	−43.20 −21.20	−23.06 ± 0.00

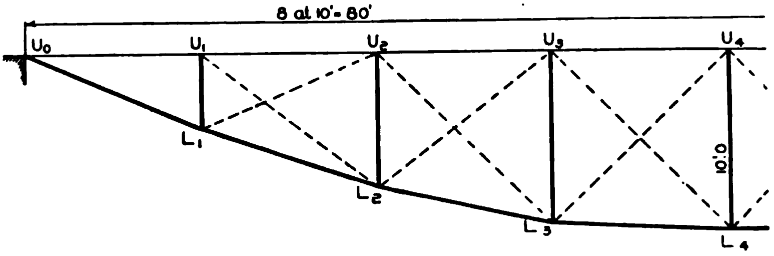


Fig. 114. Deck Parabolic Bowstring Truss.

7. In the deck parabolic bowstring truss of Fig. 114, determine the maximum stress in L_1L_2, L_1U_2, and U_3L_3. The dead panel load is 4.0, all on upper chord; and the live panel load, 20.0.

ANSWER: $L_1L_2 = +201.9$; $L_1U_2 = +21.8$; $U_3L_3 = -33.6$.

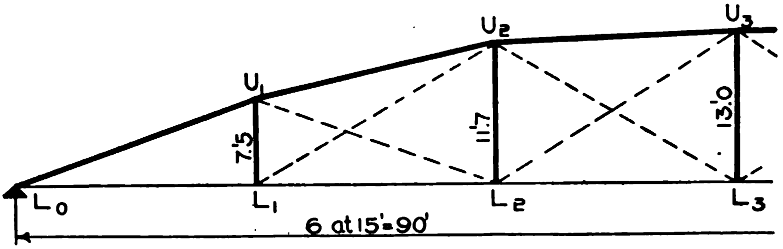


Fig. 115. Through Bowstring Truss.

8. In the through bowstring truss of Fig. 115, determine the maximum stress in U_1L_2 and L_1U_2, the dead panel load being 5.0, and the live panel load 15.0.

ANSWER: $U_1L_2 = +33.50$; $L_1U_2 = +38.0$.

9. Determine the maximum and minimum stresses in the members U_1L_1, U_1L_2, U_2L_2, and U_3L_3 of a 7-panel 175-foot through Pratt truss 30 feet high. Dead panel load is 10.0, all on lower chord; live panel load is 15.0.

ANSWER:

	U_1L_1	U_1L_2	U_2L_2	U_3L_3
d. l.	+10.0	+26.00	−10.00	0.00
l. l.	+15.0	+41.70	−21.40	−12.85
l. l.	0.0	− 2.78	+ 6.42	0.00
Max.	+25.0	+67.70	−31.40	−12.85
Min.	+10.0	+23.22	− 3.58	0.00

10. Determine the maximum and minimum stresses in the members U_1m_2, m_3L_3, U_2L_2, and m_2U_2 of the deck Baltimore truss shown in Fig. 116. Dead panel load, 30 000 lbs.; live panel load, 50 000 lbs. One-third of dead panel load is applied at the lower ends of all the verticals.

ANSWER:

	U_1m_2	m_3L_3	U_2L_2	m_2U_2
d. l.	+190.8	+ 84.8	−110.0	+21.2
l. l.	+333.5	+191.5	−211.0	+56.6
l. l.	− 15.1	− 50.5	+ 10.7	0.0
Max.	+524.3	+276.3	−321.0	+56.6
Min.	+175.7	+ 34.3	− 99.3	+21.2

11. In the truss of Problem 10, determine the maximum stress in M_2U_2 and L_3L_4.

ANSWER: $M_2U_2 = -840.0$; $L_3L_4 = +960.0$.

12. Determine the position of the wheel loads of Cooper's E 40 loading to produce the maximum positive live-load shears in the panels of a 7-panel 175-foot Pratt truss.

ANSWER: L_1, wheel 4; L_2, wheels 3 and 4; L_3, wheel 3; L_4, wheel 3; L_5, wheel 2; L_6, wheel 2.

13. Determine the maximum positive live-load shears for the truss of Problem 12.

ANSWER: $V_1 = 192.8$; $V_2 = 137.8$; $V_3 = 90.8$; $V_4 = 52.6$; $V_5 = 25.0$; $V_6 = 6.8$.

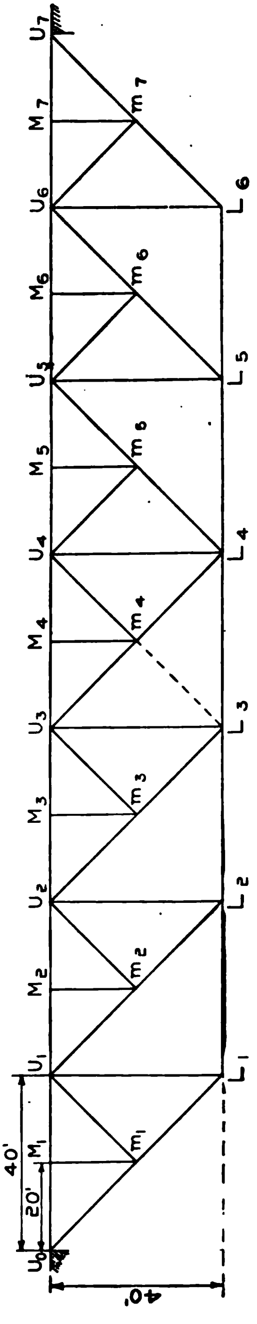


Fig. 116. Deck Baltimore Truss.

14. Determine the position of the wheel loads of Cooper's E 40 loading to produce maximum moments at the panel points of the truss of Problem 12.

ANSWER: L_1, wheel 4; L_2, wheel 7; L_3, wheels 11 and 12; L_4, wheels 13 and 14.

15. Determine the maximum moments at the panel points of the truss of Problem 12. Loading, Cooper's E 40.

ANSWER: $M_1 = 4\,820\,000$; $M_2 = 7\,745\,000$; $M_3 = 9\,192\,000$; $M_4 = 9\,082\,000$, all in pound-feet.

16. Compute the maximum live-load web stresses in the truss of Problem 12, the height being 32 feet. Loading, E. 40.

17. Compute the maximum live-load chord stresses in the truss of Problem 12, the height being 32 feet. Loading, E 40.

18. Compute the impact stresses for all members of the truss of Problem 12.

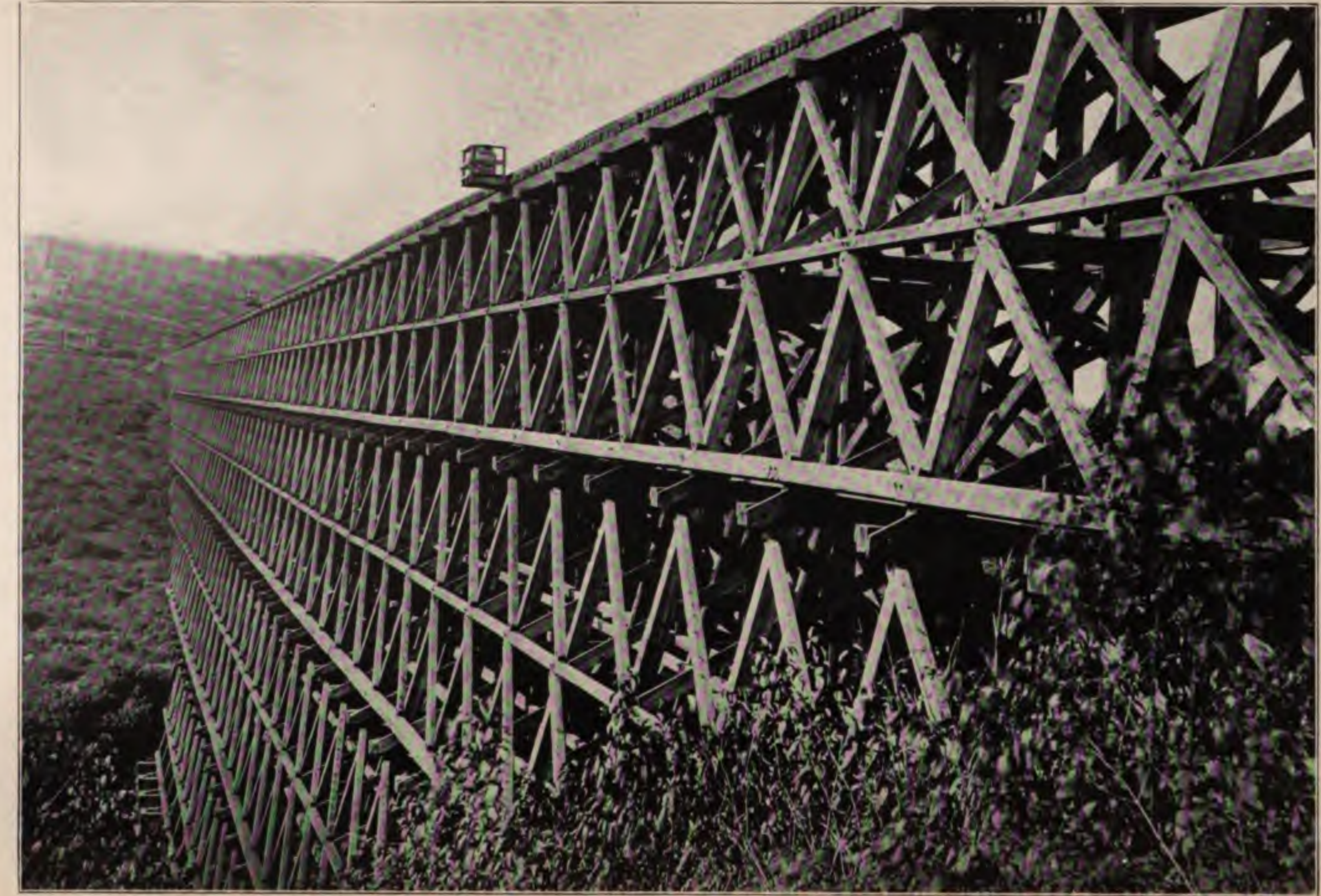
19. Determine the maximum live-load shears at the tenth-points of a 65-foot span deck plate-girder. Loading, E 40.

ANSWER: $V_0 = 103.0$; $V_1 = 86$; $V_2 = 69.7$; $V_3 = 54.5$; $V_4 = 40.8$; $V_5 = 28.4$.

20. Compute the shear due to impact in the girder of Problem 19.

ANSWER: $V_0 = 84.7$; $V_1 = 71.5$; $V_2 = 58.8$; $V_3 = 47.0$; $V_4 = 35.6$; $V_5 = 25.4$.

21. Compute the maximum live-load moments at the tenth-points of the girder of Problem 19. Loading, Cooper's E 40.



BELLEVUE TRESTLE

On line of Algoma Central Railroad, northern Ontario, Canada, spanning valley of Goulais River. A single-track bridge, 1,510 ft. long,

ANSWER:

POINT	WHEEL	MOMENT	IMPACT MOMENT
1	2	6 540	5 520
2	2	11 320	9 520
3	3	14 860	12 500
4	4	16 850	14 050
5	4	16 860	14 530
1.45′ from center	4	16 920	14 650

All moments are in thousands of pound-inches.

BRIDGE ENGINEERING

PART II

BRIDGE DESIGN

60. General Economic Considerations. The prime consideration which influences the decision to build is *cost*. After the decision to build has been made, the problem is one of a purely engineering character, whereas in the first case it was one of either a political or an engineering character, or both. The engineering problem is an economic one, in which maximum benefits must be obtained at a minimum cost.

A map of the proposed bridge site and the approaches, as well as of the country for a considerable distance up and down stream, should be made. This map should show the contours, the soundings, the borings, the high and low water-mark elevations, and the exceptional flood line. On this map the bridge should be plotted in its proposed location and also in various others. In the case of each of these locations, various schemes taking into account different numbers of piers and spans should be considered.

Several authors have attempted to present formulæ having a more or less theoretical derivation and purporting to indicate the correct number of piers and spans for a minimum cost. The use of these formulæ should not be encouraged, since they do not in any case give results close enough to serve for anything but a rough guide.

The cost of abutments will vary somewhat with the location and the character of the approach. This variation is usually small, and ordinarily an approximate location of the abutments can be quickly made. As the number of abutments is in all cases constant, their effect upon the problem of the location of the bridge is small, the main proposition being that of the cost and the number of piers and spans.

The cost of the piers will usually not be constant, those closer to the middle of the stream costing more on account of the depth of the water and the more difficult character of the foundation. Piers

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should not be placed on a skew; neither should they be placed directly in the maximum line of action of the current. If a skew is unavoidable, it should be as small as possible. The cost of piers should be ascertained by the most careful estimates. In the case of small bridges where there are only one or two piers, the matter is very simple, but with a considerable number of piers the problem becomes very complicated and requires weeks and sometimes months or years for its solution.

The determination of the cost of the superstructure is a comparatively simple matter. In certain instances the class of bridge is limited to some extent by the specifications. Cooper, in Article 2 of his "Specifications for Steel Railroad Bridges and Viaducts" (edition of 1906), gives the following:

Types of Bridges for Various Spans

SPANS	KIND OF BRIDGE
Up to 20 feet	Rolled beams
20 to 75 "	Riveted plate-girders
75 to 120 "	Riveted plate- or lattice-girders
120 to 150 "	Lattice or pin-connected trusses
Over 150 "	Pin-connected trusses

One railroad expresses a preference for plate-girders for all spans from 20 to 115 feet; and for spans from there to 150 feet, riveted trusses.

The question as to whether the bridge will be deck or through is one which is decided by the controlling influences of *water-way, false work, time of erection,* and *extra cost of masonry.* If the clear height required for the water-way is sufficiently small, the deck bridge should be chosen, as in this class the cost of false work is less, the time of erection is less, and the cost of masonry is less by an amount equal to the cross-section of the piers times the depth of the truss. Deck bridges also cost less than through bridges of equal span.

The conditions permitting, girders should be used in preference to trusses. While for equal spans girders are heavier and therefore cost more, the steel work alone being considered, little or no false work is required, and the time of erection is much less than in the case of trusses. This makes the total cost of girder bridges less than those in which trusses are used. Another item in favor of girders is their great stiffness.

While pin-connected bridges cost less and are easier to erect, their stiffness is not so great as that of riveted bridges, which cost more. The time required for the erection of riveted bridges is also greater than that for pin-connected bridges. This is on account of the great amount of time required to make the riveted connections. For long spans, say over 200 feet, it is necessary to use pin-connected bridges, as the extreme size of the connection plates prohibits the use of the riveted type. Also, it is unnecessary to use riveted long-span trusses to obtain stiffness, as the weight of the pin-connected bridges is so great when compared with the live load that sufficient stiffness is obtained.

The cost of spans of different lengths and character may be obtained directly from the bridge companies; or their weights may be computed from the formulæ given in Article 20, p. 9 (Part I, "Bridge Analysis"), and multiplied by the unit price which your experience indicates is correct, thus giving the total cost.

Evidently the solution of problems of this nature cannot be made within the limits of this text, but the following example will tend to indicate somewhat the manner of procedure in a problem of this kind. For example, if the length between abutments is 1 400 ft., the cost of each abutment is $12 000, and the cost of each pier is $15 000, then, if we have fourteen 100-foot plate-girder spans, each costing $4 300, and thirteen piers, the total cost will be $279 200. On the other hand, if nine piers and ten 140-foot truss spans, each costing $9 200, are used, the cost will be $251 000, showing a balance of $28 200 in favor of the truss scheme. The live loading is E 50.

61. Economic Proportions. The depth of girders is given in Article 59, Part I.

In the case of trusses, the effect of an increase in the height is to increase the stresses in the web members and to decrease the stresses in the chord members. This variation does not affect the weights to any considerable extent; in fact, a variation of 20 per cent in the height will not affect the weight more than 2 or 3 per cent.

The height of the bridge is usually fixed by some considerations which are in turn determined by the specifications. The height must be sufficient to clear whatever traffic will pass through. It should also be sufficient to prevent overturning on account of the wind pressure on the truss or on the traffic. In addition, the height of the bridge is influenced by the depth of the portal bracing. A deep portal bracing is desirable, in that it stiffens the trusses under the action of the wind and the vibration due to the passing traffic; but a deep portal bracing increases the height of the truss and therefore the bending in the end-posts due to the wind. Judgment on the part of the engineer should be used in order to determine the limiting height for securing a maximum amount of benefit as regards stiffness and a minimum amount of bad effect due to the bending in the end-posts. Fig. 117, which gives the height for any given length of span, may be said to represent the best modern practice (1908). Variations of a foot or more from those given do not affect the weight to any appreciable extent.

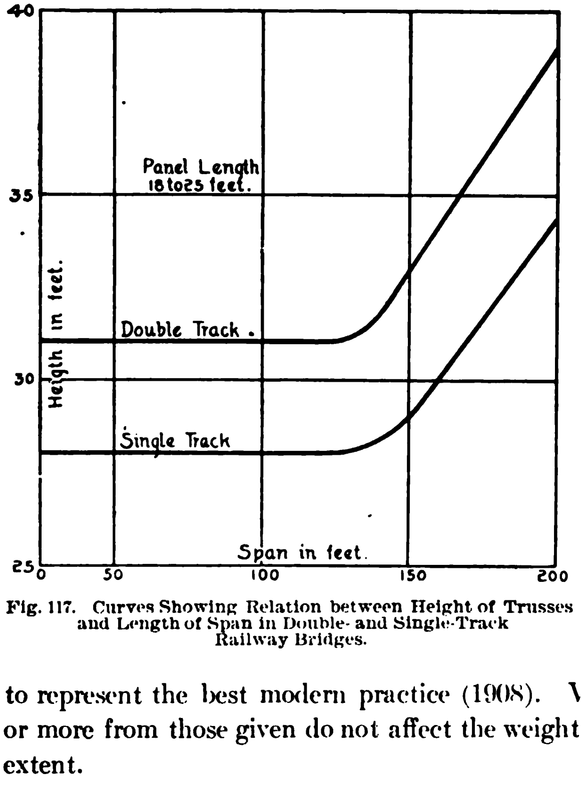


Fig. 117. Curves Showing Relation between Height of Trusses and Length of Span in Double- and Single-Track Railway Bridges.

The distance from center to center of trusses for highway bridges depends upon the width of the street or, if in the country, the width of the roadway. Streets, of course, vary in width in different localities, but country highway bridges usually have a roadway of from 14 to 16 feet in the clear.

In the case of railroad bridges, the distance from center to center of trusses depends upon whether the track is straight or on a curve, and also upon whether the bridge is a deck or a through bridge.

The actual amount varies in most cases, and is fixed by specification. Some specifications require that when the track is straight, the distance from center to center of trusses shall be 17 feet; or that, in case one-twentieth of the span exceeds the 17 feet, then one-twentieth of the span shall be used.

For deck plate-girders the common practice appears to be to space them as given below:

Width of Plate-Girder Bridges for Various Spans

Spans	Distance Center to Center of Plate-Girders
Up to 65 feet	6 feet 6 inches
65 to 80 "	7 feet 0 inches
80 to 115 "	7 feet 6 inches

For through plate-girders the spacing should be such that no part of the clearance diagram will touch any part of the girder. In case of double-track plate-girders with one center girder, great care should be exercised in order that the center girder shall not be so deep nor have so wide a flange as to interfere with the clearance diagram (see Fig. 126).

On account of the wind on a train which runs on track placed at the elevation of the top chord of deck bridges, the overturning effect is exceedingly great, and special care should be taken that the height and width are such as to prevent overturning.

In through bridges the clearance must be such as to allow the clearance diagram to pass. Special attention should be paid to the knee-braces and also to the portal braces. When the bridge is on a tangent, the spacing of the trusses is a comparatively simple matter, being just sufficient for the clearance diagram; but on curves, allowance must be made for the tilt of the diagram due to the superelevation of the outer rail, and also allowance must be made for the fact that the length of the cars between trucks forms a chord to the curve, and as such the middle ordinance must be taken into account. It is also necessary to allow for that part of the car which projects over the trucks, as this will extend beyond the outer rail by an amount greater than one-half the width of the clearance diagram. (See Figs. 119 and 120.)

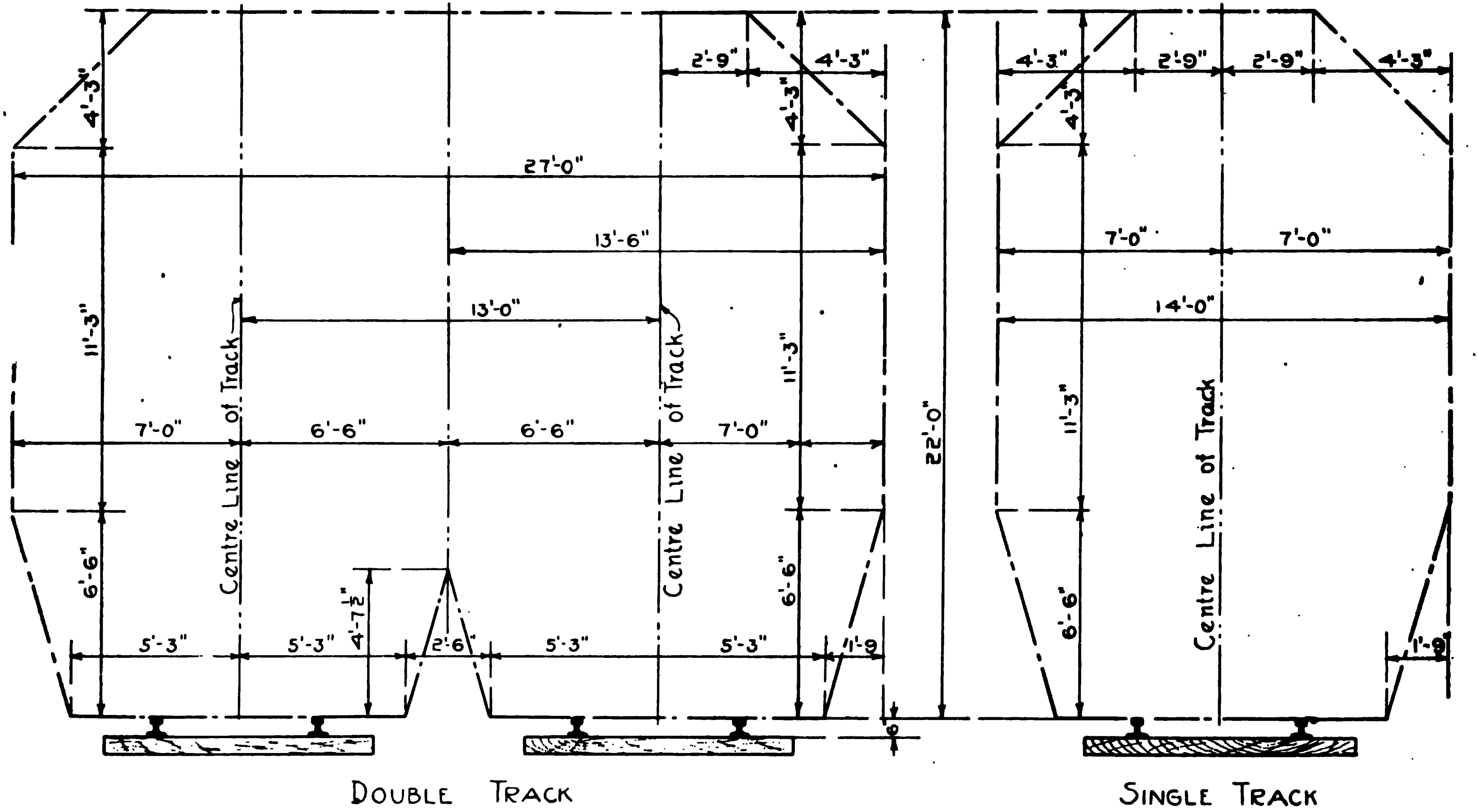


Fig. 118. Standard Clearance Diagram (Straight Track) Used by Lehigh Valley Railroad. Courtesy of F. E. Schall, Bridge Engineer.

Dimensions given show standard clearance for straight tracks. When tracks are on a curve, equivalent clearance is to be provided, assuming length of cars 75 feet over all and 54 feet center to center of trucks. Also, on account of tipping of cars, for structures extending over 12 feet 6 inches above top of rail, an additional clearance of 2½ inches is to be provided on the inside of curves for each inch of elevation of outer rail. For structures of lesser height, proportionate allowance is to be made.

62. **The Clearance Diagram.** The clearance diagram is not supposed to represent the outline of the largest engine or car which may run over the line, but represents the maximum amount of space which may be taken up by objects which are to be shipped over the line. For instance, the lower part of the clearance diagram may allow for snow-plow or ballast distributors, and the upper part may take into account the passage of such material as carloads of lumber, piles, or telegraph poles. The standard clearance diagram of the Lehigh Valley Railroad is given in Fig. 118. This diagram is for the clearance on straight track only. On curves, the diagram tilts as shown in Fig. 119, and to allow for this tilting the Lehigh Valley Railroad requires 2½ inches additional clearance on the inside of curves for each inch of elevation of the outer rail. In addition to this tilting effect, the clearance should also be increased on account of the length of the cars and their projection over the outer and inner rails. Fig. 120 shows a standard car according to the specifications of the American Railway Engineering & Maintenance of Way Association, in such a position on a single-track span as to show the effect of the curve upon the widening of the spacing, center to center of trusses.

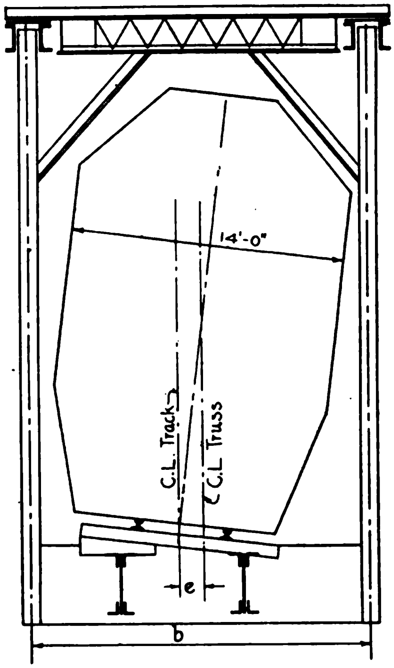


Fig. 119. Clearance Diagram on Curves, Showing Tilting.

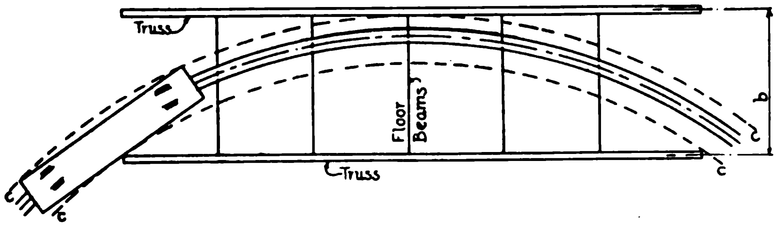


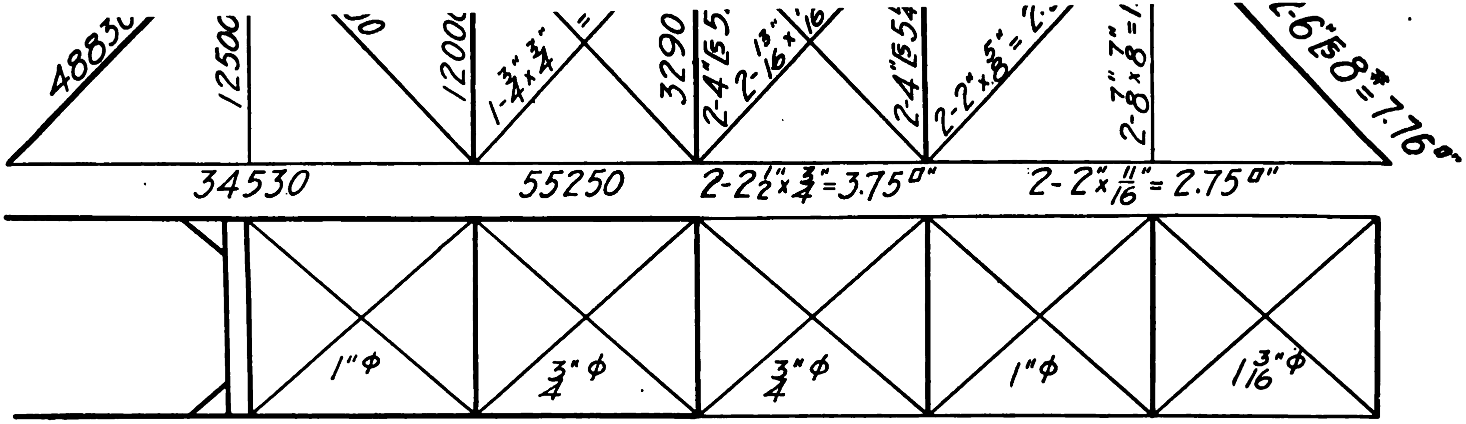
Fig. 120. Standard Car on Curve, Showing Necessity for Wider Spacing of Trusses.

This car is 80 feet long, 60 feet between centers of trucks, and is as wide as the clearance diagram, 14 feet for single track. It is evident that the trusses cannot be spaced so as to interfere with the clearance line of the body of the car and its projecting ends. These clearance lines are represented as broken lines in Fig. 120, and are marked c–c. Note that the center of the track is seldom in the center of the floor-beam. Also, it is evident that the sharper the curve, the greater the required distance between trusses, and accordingly the greater the floor-beams in length. This varies the moment in the different floor-beams and therefore makes them more costly. The stringers, also, are more costly, on account of the fact that their ends are skewed. On account of the eccentricity of the track, one truss takes more of the load than the other, and therefore the trusses are not the same—a fact which further increases the cost.

From the above it is seen that almost all conditions incident to the building of a bridge on a curve tend to increase the cost; and hence a fundamental principle of bridge engineering: *Avoid building bridges on curves.*

63. **Weights and Loadings.** For the weight of steel in any particular span, and for the loading required for any particular class of bridge, see Articles 20 to 23, Part I. The weight of the ties and the rails and their fastenings is usually set by the specifications at 400 pounds per linear foot of track. For highway bridges the weight of the wooden floor is usually taken at 4½ pounds per square foot of roadway for every inch in thickness of floor.

Highway bridges are divided into different classes according to their loadings (see Cooper's Specifications). The decision as to the class to be employed depends somewhat upon the distance to the nearest bridge across the same stream. In case the nearest bridge is only a few miles away and is of heavy construction, it is not actually necessary to construct a heavy bridge at the proposed site, the heavier traffic being required to pass over the other bridge. In case a heavy bridge is not in the neighborhood, then one should be constructed at the proposed site. If the proposed site is on a road connecting adjacent towns of large size, then a heavy bridge should be constructed and provision made for future interurban traffic, even if none is at that time in view, since it will be more economical to do this than to erect a new bridge in the future.



Length of Span, C. to C. 102'-0"
Clear Width of Roadway, 12'-0
6 Panels, 17'-0" Each
Height of Truss, C. to C. of Pins, 17'-0"
Portal Bracing, 4Ls $2\frac{1}{2}$ x 2" x 3.7#
Top Lateral Struts, 4Ls 2" x 2" x 3.2#
Sway Struts – Sway Rods –
Shoe Struts, 4" I $7\frac{1}{2}$# Knee Braces, 3" x 3" x $\frac{1}{4}$" L

Bottom Lateral Ties, $1\frac{3}{16}$"φ, 1"φ, $\frac{3}{4}$"φ
Top Lateral Ties, 1"φ, $\frac{3}{4}$"φ
Floor Beam, 18" x $\frac{1}{4}$" & 4Ls 3" x 3" x 4.5#
Sidewalk Brackets, —
Joists, 6 Lines 3" x 12"
Flooring, $2\frac{1}{2}$" Plank, Joist Bearers, 4x4
Wheel Guards, $2\frac{1}{2}$" x 8" Spiking Pieces, —
Hub Guards, 2 Lines $1\frac{1}{4}$" Gaspipe

Live Load 100 Lbs. per Sq. Ft. of Floor

Plate I. Stress Sheet of a Highway Bridge.

In the case of railroad bridges, new ones are nearly always constructed to carry the heaviest main line engines. These are usually of a class corresponding to Cooper's E 40 or E 50. In some localities branch-line bridges are built for the same live loadings; but in the majority of cases the branch-line bridges consist of the old bridges from the main line.

64. **Specifications.** For any particular bridge the specifications are either written by the engineer in charge, or some of the very excellent general specifications which are on the market in printed form are used. Some railroads use these general specifications with the addition of certain clauses which are desired by the chief or bridge engineer. The principal differences in these general specifications are in regard to the allowance for impact.

Whenever highway design is mentioned in this text, it is to be in accordance with Cooper's Highway Specifications (edition of 1901). Wherever plate-girder design is given, it is in accordance with Cooper's Railway Specifications (edition of 1906); and wherever truss design is given, it is in accordance with the general specifications of the American Railway Engineering & Maintenance of Way Association (second edition, 1906).

65. **Stress Sheet.** Before the sections are designed, the computer makes a skeleton outline of the truss, and on this places the dead-load and live-load stresses, and, in case the wind should be considered, the wind-load stresses. This is sent to the designer. The designer determines the various sections, and also the moments and shears in the stringers and floor-beams. These are placed on a sheet usually 17 by 23 inches. This is called a *stress sheet.* This sheet is now given to the draftsman, who makes a shop drawing. The stress sheets for railroad bridges are usually more elaborate than those for highway bridges. Plate I is the stress sheet of a highway bridge; and Plate II (Article 78) and Plate III (Article 93) are examples of the best modern practice in the making of plate-girder and truss-bridge stress sheets.

66. **Floor System.** Perhaps no part of bridge design is better standardized than the construction of the open steel floors for railroad bridges. The stringers are usually placed 6 feet 6 inches apart, and consist of small plate-girders, or, if the panel length is short, of one or more I-beams. I-beams are economical in regard to

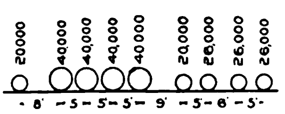
TABLE XIX

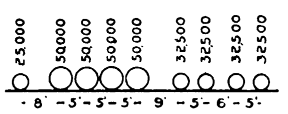
Safe Spans for I-Beams

(Based on unit-stress of 10 000 lbs. per square inch in extreme fibre)

Size of I-Beam	Weight per Foot (Lbs.)	Moment of Inertia 1 I-Beam	Engine Class E 40			Engine Class E 50		
			Safe span C to C of Bearings			Safe span C to C of Bearings		
			1 Beam per rail	2 Beams per rail	3 Beams per rail	1 Beam per rail	2 Beams per rail	3 Beams per rail
9 in.	25	92	....	5 ft. 3 in.	7 ft. 9 in.	....	4 ft. 9 in.	7 ft. 0 in.
9 "	30	102	....	5 " 9 "	8 " 9 "	....	5 " 0 "	7 " 0 "
9 "	35	112	....	6 " 6 "	9 " 6 "	....	5 " 3 "	7 " 9 "
10 "	30	135	3 ft. 6 in.	6 " 9 "	10 " 0 "	3 ft. 0 in.	5 " 9 "	8 " 6 "
10 "	35	163	4 " 3 "	8 " 6 "	11 " 0 "	3 " 6 "	7 " 0 "	10 " 0 "
10 "	40	175	4 " 6 "	9 " 0 "	12 " 0 "	3 " 9 "	7 " 6 "	11 " 0 "
12 "	31½	218	4 " 9 "	9 " 3 "	12 " 9 "	4 " 0 "	7 " 9 "	11 " 3 "
12 "	40	274	5 " 9 "	11 " 6 "	13 " 9 "	4 " 9 "	9 " 6 "	12 " 3 "
12 "	50	332	7 " 3 "	12 " 6 "	15 " 3 "	6 " 0 "	11 " 3 "	13 " 6 "
12 "	65	403	8 " 6 "	13 " 9 "	16 " 0 "	7 " 0 "	12 " 3 "	15 " 0 "
15 "	42	443	7 " 6 "	12 " 9 "	15 " 6 "	6 " 3 "	11 " 6 "	14 " 0 "
15 "	50	515	8 " 6 "	13 " 9 "	16 " 3 "	7 " 3 "	12 " 6 "	15 " 3 "
15 "	60	619	10 " 6 "	15 " 3 "	19 " 0 "	8 " 9 "	13 " 6 "	16 " 9 "
15 "	70	718	11 " 6 "	16 " 6 "	20 " 9 "	10 " 3 "	14 " 9 "	18 " 6 "
15 "	80	774	12 " 0 "	17 " 3 "	21 " 6 "	10 " 9 "	15 " 3 "	19 " 3 "
18 "	55	809	11 " 0 "	16 " 0 "	20 " 0 "	9 " 6 "	14 " 0 "	17 " 9 "
18 "	65	890	11 " 9 "	16 " 9 "	20 " 9 "	10 " 6 "	15 " 0 "	18 " 9 "
18 "	75	1 023	12 " 6 "	18 " 0 "	22 " 6 "	11 " 6 "	16 " 0 "	20 " 0 "
18 "	80	1 063	12 " 9 "	18 " 9 "	23 " 0 "	11 " 9 "	16 " 6 "	20 " 6 "
18 "	90	1 188	13 " 6 "	19 " 6 "	24 " 6 "	12 " 3 "	17 " 6 "	21 " 9 "
20 "	65	1 180	12 " 9 "	18 " 9 "	23 " 0 "	11 " 6 "	16 " 6 "	20 " 9 "
20 "	75	1 277	13 " 3 "	19 " 6 "	24 " 0 "	12 " 0 "	17 " 3 "	21 " 3 "
20 "	85	1 453	14 " 3 "	21 " 0 "	26 " 0 "	12 " 9 "	18 " 6 "	23 " 0 "
20 "	90	1 502	14 " 6 "	21 " 3 "	26 " 6 "	13 " 0 "	18 " 9 "	23 " 3 "
20 "	100	1 650	15 " 3 "	22 " 3 "	28 " 0 "	13 " 6 "	19 " 9 "	24 " 6 "
24 "	80	2 112	15 " 9 "	23 " 0 "	29 " 3 "	14 " 0 "	20 " 6 "	25 " 9 "
24 "	85	2 182	16 " 0 "	23 " 9 "	29 " 9 "	14 " 6 "	21 " 0 "	26 " 3 "
24 "	90	2 356	17 " 0 "	24 " 6 "	30 " 9 "	14 " 9 "	21 " 6 "	27 " 3 "
24 "	95	2 427	17 " 3 "	25 " 0 "	31 " 6 "	15 " 0 "	22 " 0 "	27 " 9 "
24 "	100	2 497	17 " 6 "	25 " 3 "	32 " 0 "	15 " 3 "	22 " 6 "	28 " 0 "

DIAGRAM OF ENGINES





first cost, but are disadvantageous on account of the eccentric connections which necessitate heavy brackets to resist part of their reaction. They are also somewhat undesirable on account of the fact that, the ties deflecting, most of the load is carried by the inner I-beam. However, I-beams for stringers and for short-span bridges (see Fig. 121) are much used in present practice, and give good results. Figs. 121 to 127 show the standard open floor sections of the Lehigh Valley Railroad. Table XIX gives the required number of I-beams, together with their weight, which are to be used for short-span bridges or as stringers in panels of given length.

Solid floors consist of angles and plates, channels and plates, or other shapes. They extend transversely across the bridge from truss

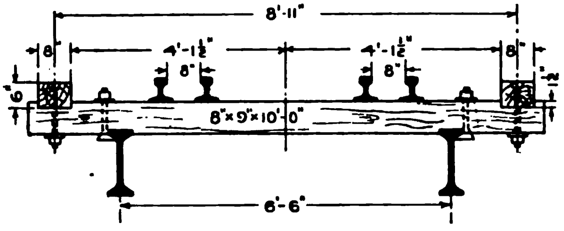


Fig. 121. Sectional View Showing Open-Floor Construction of Railroad Bridge of Short Span, Single Track. I-Beams used for Stringers. Lehigh Valley Standard.

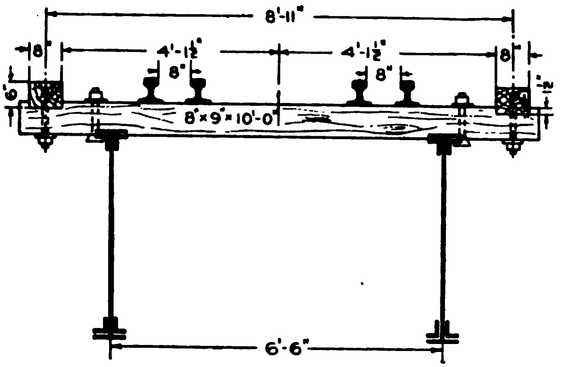


Fig. 122. Section of Open-Floor Construction of Deck-Girder and Truss Bridge, Single Track. Lehigh Valley Standard. Plate-Girders used for Stringers.

to truss, the lower chords, in case of truss bridges, being made heavy enough to act as girders as well as tension members. Figs. 128 to 130 show sections of solid floors. The ballast is laid directly upon these solid floors, which are first covered with a good damp-proof paint. The floors should also be supplied with good drainage facilities.

Concrete is sometimes laid directly upon the steel floor, and the ballast put upon this concrete,

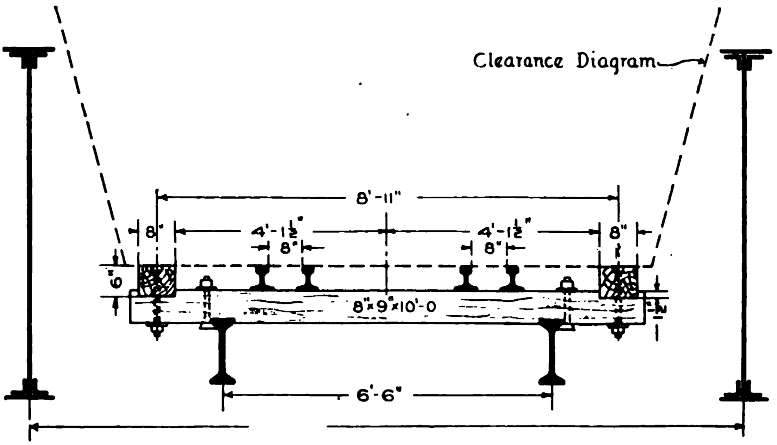


Fig. 123. Floor Construction of a Through-Girder Bridge, Single Track. Lehigh Valley Standard.

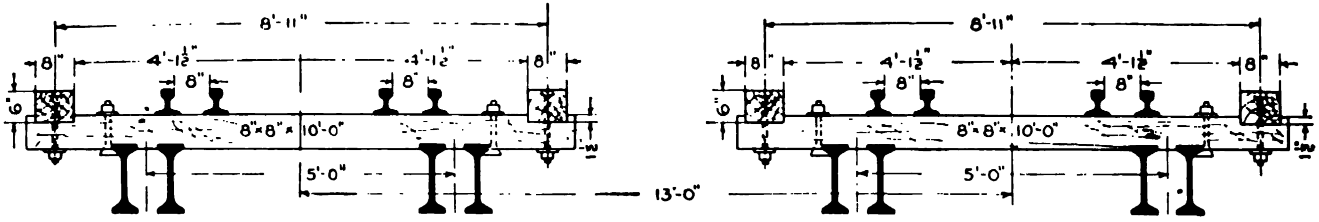


Fig. 124. Floor Construction of Double-Track I-Beam Bridge, Lehigh Valley Standard.

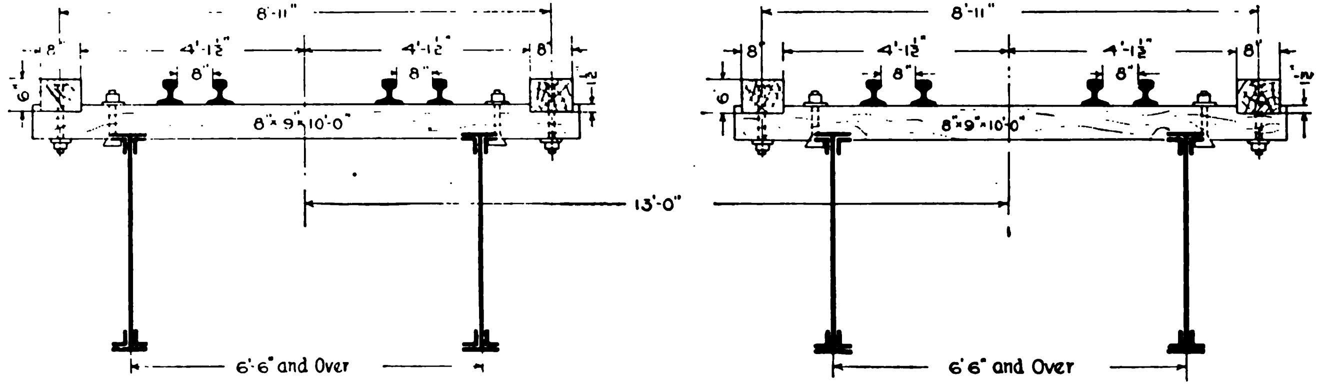


Fig. 125. Floor Construction of Double-Track Deck-Girder and Truss Bridge, Lehigh Valley Standard.

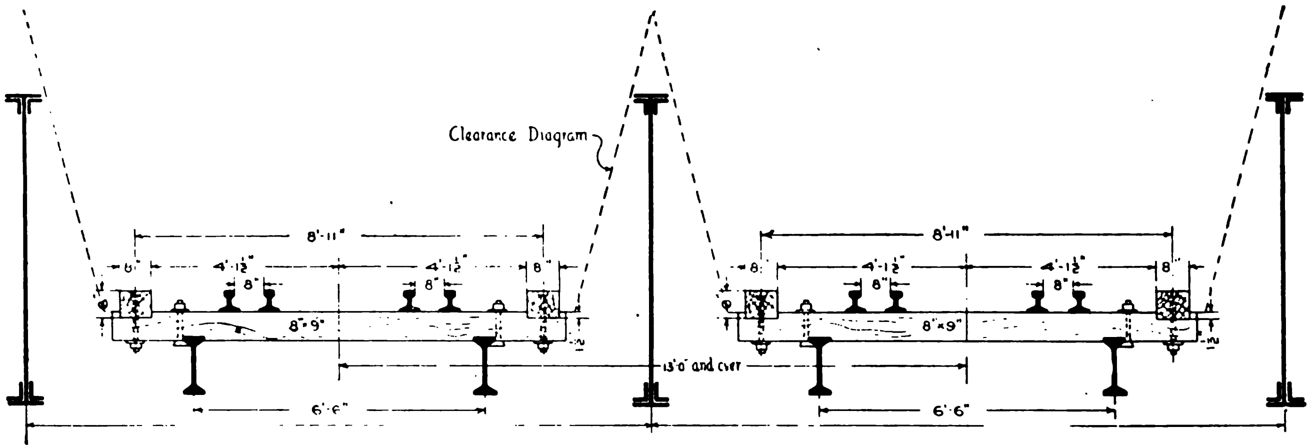


Fig. 126. Floor Construction of Double-Track Through-Girder Bridge, Lehigh Valley Standard.

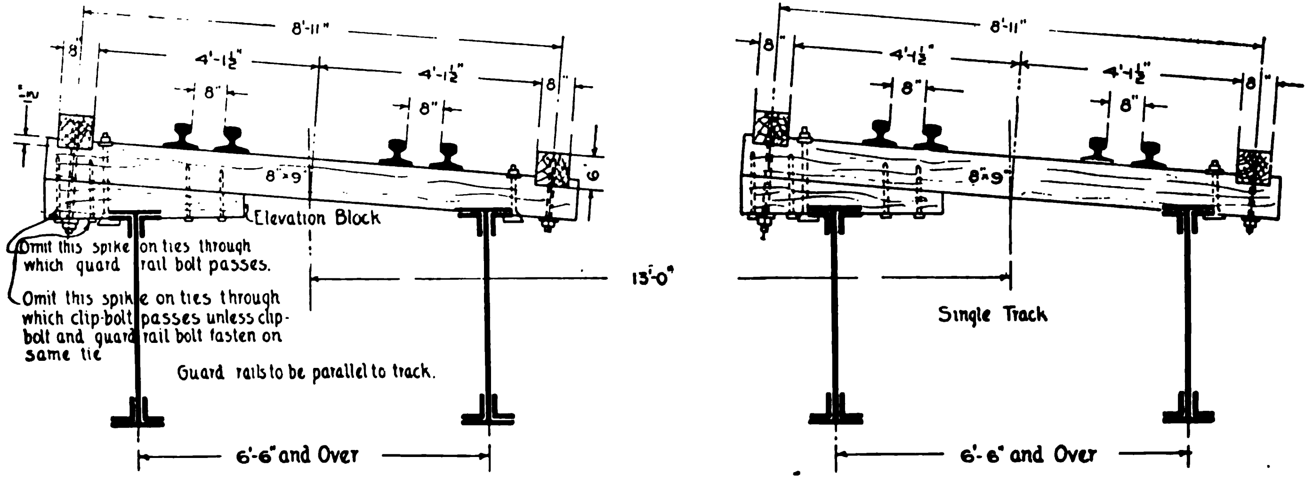
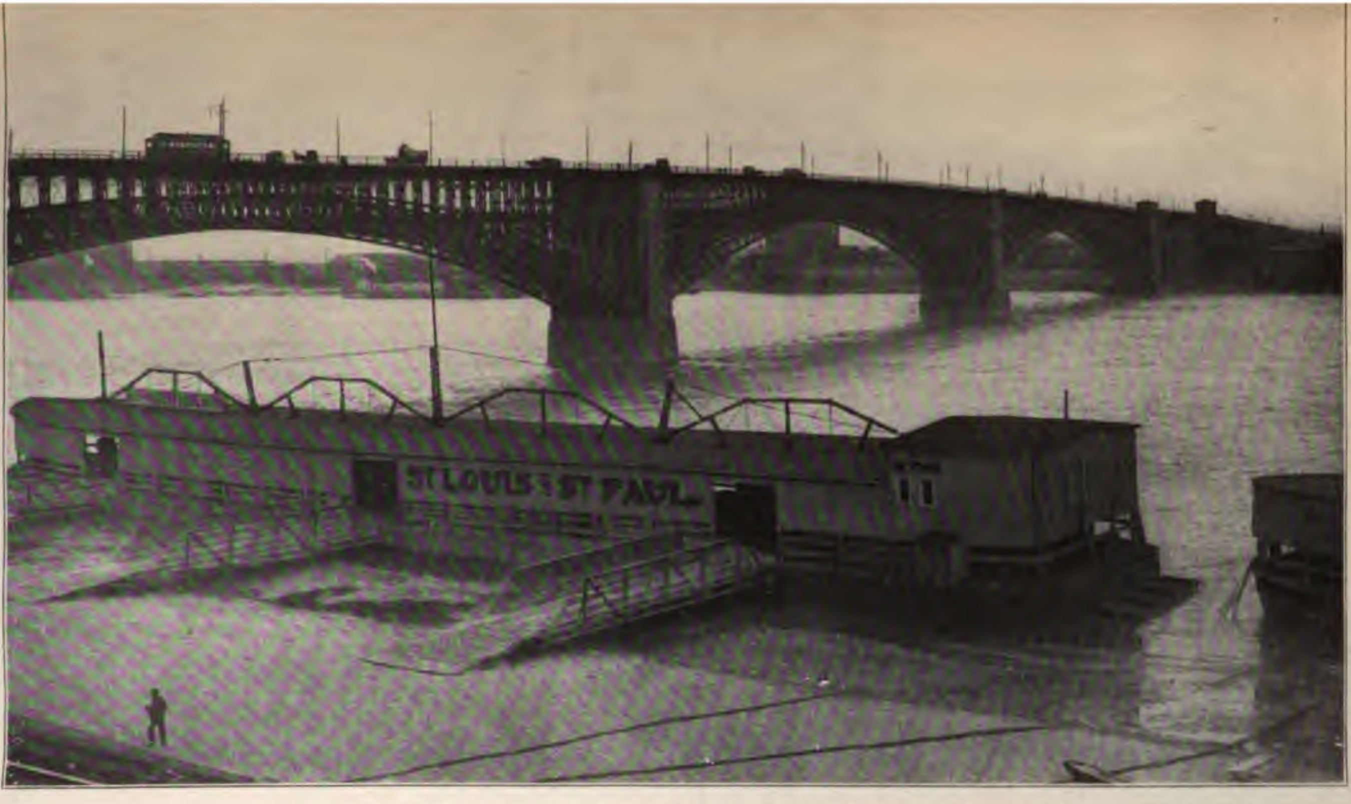

8'-11"
4'-1½"
8"
8"×9"
6"
Elevation Block
Omit this spike on ties through which guard rail bolt passes.
Omit this spike on ties through which clip-bolt passes unless clip-bolt and guard rail bolt fasten on same tie
Guard rails to be parallel to track.
13'-0"
Single Track
6'-6" and Over

Fig. 127. Floor Construction of Double-Track Deck-Girder and Truss Bridge on Curve, Lehigh Valley Standard.



THE EADS BRIDGE OVER THE MISSISSIPPI RIVER AT ST. LOUIS, MISSOURI

A steel arch viaduct for railway and highway traffic, trains running beneath the roadway. Built by James B. Eads in 1867-74. Between approaches, the bridge consists of 3 spans, 497, 515, and 497 ft. In each span are four arches composed of steel tubes 9 inches in external diameter, connected by lateral bracing. The tube sections between joints are straight, curvature being secured by inserting wedge-like plates.

Photo by Strauss.

which has previously had a layer of some good waterproofing applied on its upper surface.

67. Practical Considerations. The possibilities of the rolling mill and the various shops of a bridge company, such as the drafting room, forge, foundry, templet shop, assembling shop, and riveting and finishing shop, and also the shipping and erecting facilities, should be well known in order to make the most economical use of them. This requisite knowledge comes only from experience. The best way to obtain this experience without being actually employed in the shops, is to go into the shops every chance that presents itself, keep your eyes and ears open, and ask all the questions you can. The use to be made of handbooks of the various steel manufacturers is given in Part I of "Steel Construction," and should be thoroughly studied before going further. Some one of these handbooks is indispensable to persons designing steel structures. That of the Carnegie Steel Company (edition 1903) is one of the best, and will be frequently referred to in the present text.

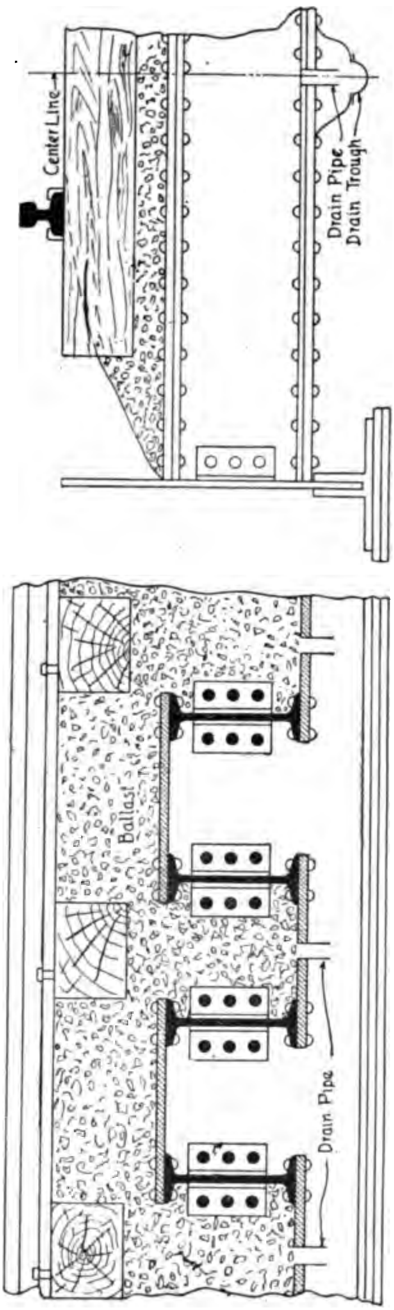


Fig. 128. Longitudinal and Transverse Section of Solid-Floor Construction.

Copies may be procured from the Carnegie Steel Company, Frick Building, Pittsburg, Pa. The usual price to students is 50 cents, to others $2.00.

DESIGN OF A PLATE-GIRDER RAILWAY-SPAN

68. **The Masonry Plan.** In some cases the general dimensions of the masonry are limited; such a case, for example, would occur in the crossing of a street or narrow waterway. Here the length of the span and the distance above the street or the surface of the water, are the limited dimensions. The span and under-clearance may be unlimited, as in the case of a country stream crossed by a roadway which is a considerable distance above the surface of the water. The term *unlimited* is not here used in its exact meaning, as the span in this case is really limited by the cost, which rapidly increases with the length of the span.

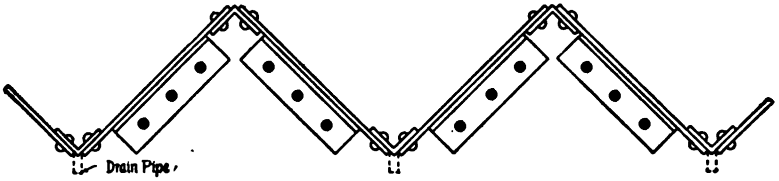


Fig. 129. Solid-Floor Construction of Plates and Angles.

In some cases, as when the engineer is in a bridge office, the masonry plans are sent in by the railroad. In such cases many of the limited dimensions are fixed. The most usual dimensions to be fixed are the elevation of base of rail, the elevation and size of the bridge seat, and the length of the span under coping. These limit the depth of the girder, or the depth of the floor if it be a through girder, and also limit the length of the bearing plates at the end. Fig. 131, the masonry plan of a road crossing, shows in general what can be expected. All the dimensions usually fixed are given, and those marked x and y may or may not be, but x should never be less than 3 feet.

69. **Determination of the Class.** As before mentioned, the deck plate-girder should be used if possible, since its cost is less. There are some cases, however—such as track elevation in cities—where the additional cost required to elevate the track so as to use a

deck plate-girder will more than balance the saving in its favor. In such cases the through plate-girder is used.

The case whose design is under consideration will be taken similar to that of Fig. 131, and the span will therefore be a deck one.

70. Determination of the Span, Center to Center. Fig. 132 shows the various spans—namely, *under coping, center to center of end bearings,* and *over all.* The span under coping is that span from under coping to under coping lines of the abutments, and is so chosen as to give the required distance between the abutments at their base. The span center to center is equal to the span under coping *plus* the length of one bearing plate. The span over all is the extreme length of the girder. The length of the bearing plate is influenced by the width of the bridge seat, and also by the maximum reaction of the girder. The length should seldom be greater than 18 inches and never greater than 2 feet, as the deflection of the girder will cause a great amount of the weight to come on the inner edge of the bearing plate and also on the masonry, in which case the masonry is liable to fail at that point and the bearing plates are over-stressed.

Cast-steel bearings are now almost universally used. They decrease the height of the masonry, and distribute the pressure more evenly and for a greater distance over the bridge seat. When these castings are used, the bearing area between them and the girder may be made quite small, thus doing away to a great extent with the

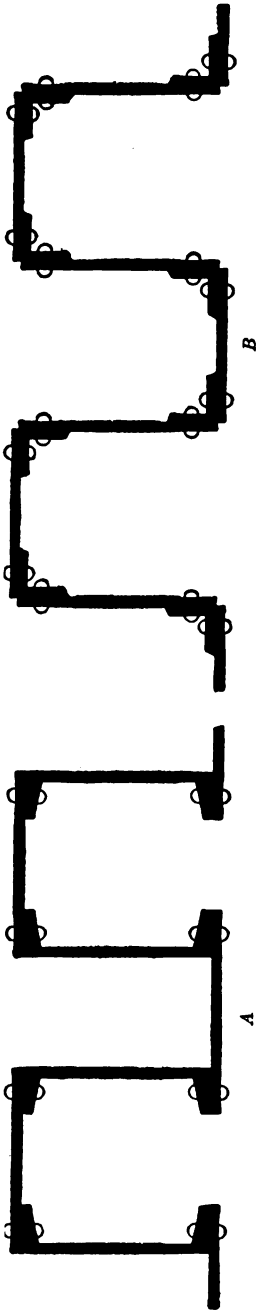


Fig. 130. Methods of Solid-Floor Construction. *A*—Plates and Channels; *B*—Plates and Angles.

deteriorating effect due to the deflection of the girder as mentioned above. Fig. 133 shows the end of a girder equipped with a cast-steel pedestal. Table XX gives the length of the bearing on the

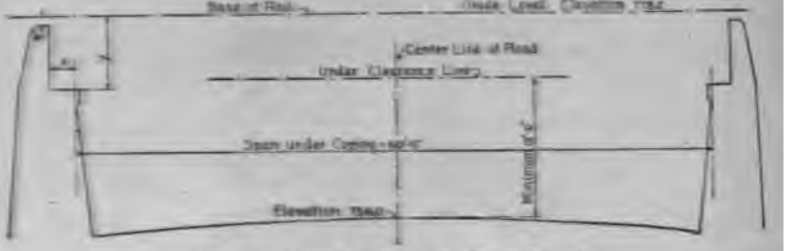


Fig. 131. Masonry Plan of a Road Crossing.

masonry for various spans, Cooper's E 40 loading being used and cast-steel pedestals being employed.

TABLE XX
Length of Masonry Bearings

Span	Length of Bearing
15 to 24 feet	12 inches
25 to 44 "	16 "
45 to 69 "	21 "
70 to 79 "	23 "
80 to 89 "	29 "
90 to 115 "	36 "

As an example, let it be required to determine the span center to center of a deck plate-girder of 60-foot span under coping, the

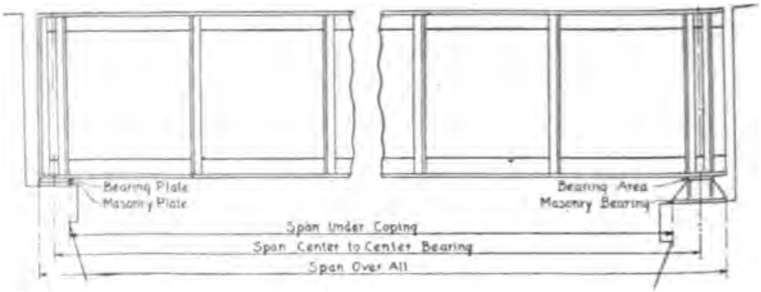


Fig. 132. Diagram Showing Various Spans Considered in Bridge Construction.

loading being E 40. From Table XX it is seen that the length of the masonry bearing will be 21 inches, and therefore the span center to center of bearings will be 60 + 2 × ($\frac{1}{2}$ × 1 ft. 9 in.) = 61 feet 9 inches.

In Articles 71 to 77 the above girder will be designed; and also such information as is of importance regarding the subject-matter of each article will be treated. The dead- and live-load shears are computed by the methods of Part I, and are given in Fig. 134.

71. **Ties and Guard-Rails.** The length of ties for single-track bridges is 10 feet. For double-track bridges the length is in most cases the same. In some double-track bridges, however, either each tie or every third tie extends entirely across the bridge. In other cases every third tie on one track extends to the opposite track, thus acting as a support for the foot-walk which is laid upon them. It is the best practice to limit the length of the ties on double-track bridges to 10 feet, since, if they extend into the opposite track in any way whatsoever, unnecessary expense is incurred whenever repairs or renewals are made, because both tracks must necessarily be disturbed to some extent.

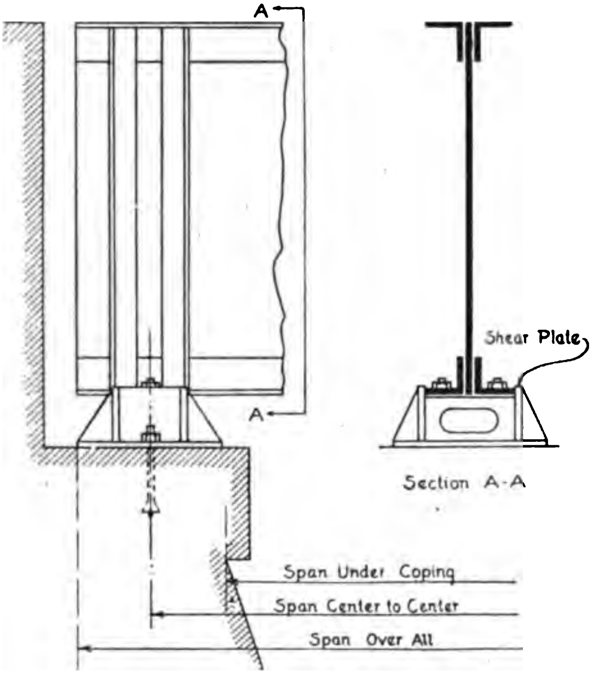


Fig. 133. End of a Girder Equipped with a Cast-Steel Pedestal.

The size of the ties varies with the weight of the engines and the spacing of the stringers or girders on which they rest. They are usually sawed to size instead of hewn, and the following sizes may be purchased on the open market—namely, 6 by 8, 7 by 9, 8 by 9, 9 by 10,

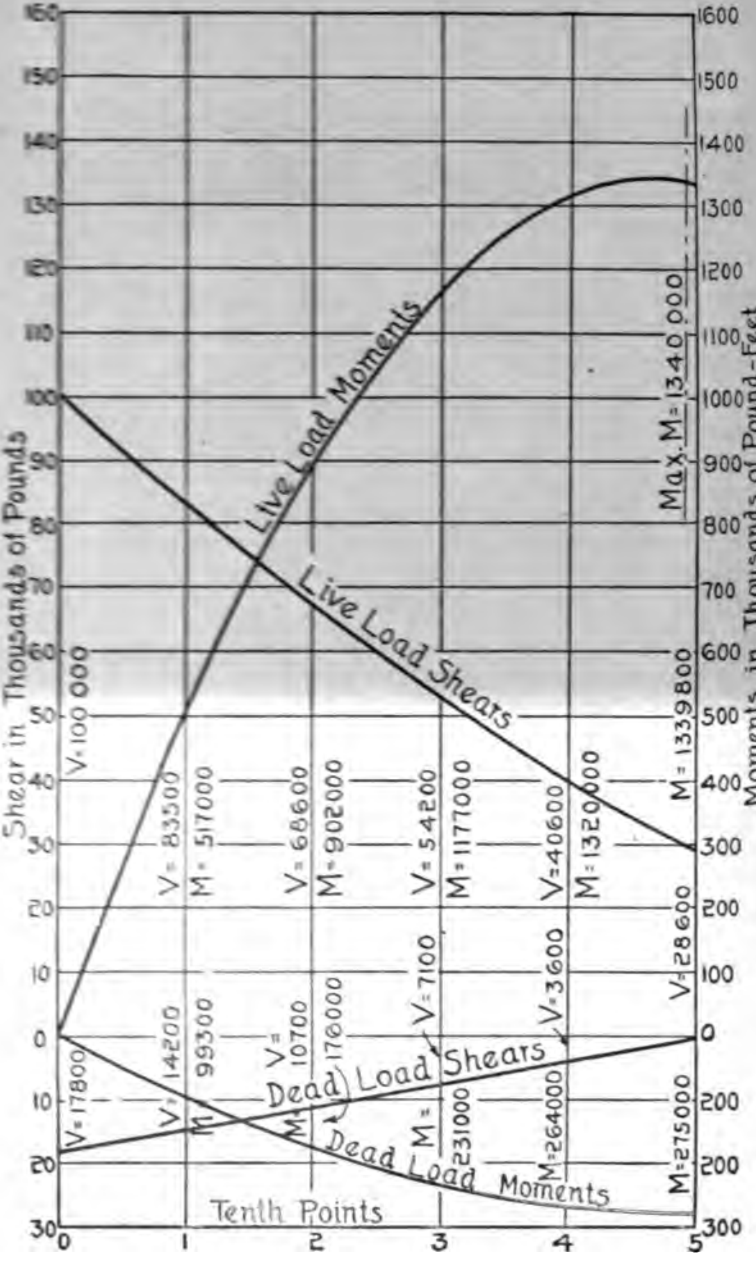


Fig. 134. Shear and Moment Diagram.

and 10 by 12 inches. Larger sizes may be obtained on special order.

The elevation blocks (see Fig. 127) should be of length to suit the width of the cover-plates and the spacing of the supports. They are usually made of the best quality of white oak, since the cost of renewal is great enough to demand that they be made of material as permanent as possible.

The guard-rails should be placed in accordance with the specifications (see Articles 13 and 14). Some railroads specify that the guard-rails shall be in 24-foot lengths unless the bridge is shorter than 24 feet, in which case one length of timber should be used. For method of connection and other details, consult Figs. 121 to 127. The guard-rails and the ties are usually made of Georgia long-leaf yellow pine, prime inspection. Other wood, such as chestnut, cedar, and oak, may be used.

In addition to the wooden guard-rail, a steel guard-rail usually consisting of railroad rails is placed within about 8 inches of the track rail.

In designing ties, the problem is that of a simple beam symmetrically loaded with two equal concentrated loads, the weight of the rail and tie itself usually being neglected. For the case in hand, which is that of a deck plate-girder, loading E 40, the concentrated load for which the tie must be designed is, according to Specifications (Article 23, 3d part), 8 333 pounds. According to Article 23, 100 000 pounds is on four wheels. This gives 25 000 pounds on one wheel, and according to Article 15, one-third of this, or 8 333 pounds, will come on one tie. Fig. 135 shows the condition of the loading, the space center to center of rail being taken as 4 feet 10 inches. Some designers take this distance as 5 feet; but as the standard rail head is about 2 inches, and the standard gauge 4 feet $8\frac{1}{2}$ inches, the distance here taken seems to be the more logical one.

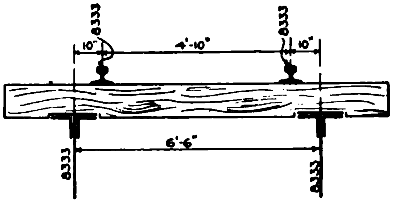


Fig. 135. Distribution of Loading on Ties of Deck Plate-Girder Bridge.

The formula to be used in the design of this beam is that given in "Strength of Materials," and is $M = \frac{SI}{c}$. In this case $M = 10 \times$

[illegible] = 83 160 pound-inches. In the above formula, $I = bd^3 \div 12$, and $c = d \div 2$, and therefore $\frac{I}{c} = \frac{bd^2}{6}$. Substituting the value of the moment in the above formula, and solving for S, there results:

$$S = \frac{498\,960}{bd^2}$$

For a 6 by 8-inch tie, the unit-stress would then be:

$$S = \frac{498\,960}{6 \times 8^2} = 1\,300 \text{ pounds.}$$

If a 7 by 9-inch tie is used, the unit-stress is found to be 880 pounds. Since according to Article 15 of the Specifications, the unit-stress cannot be greater than 1 000 per square inch, it is necessary to use a 7 by 9-inch tie. If the engine loading had been E 50, the moment would have been 100 000 pound-inches, and then the stress in a 7 by 9-inch tie would be 1 060 pounds per square inch, and the stress in an 8 by 9-inch tie would be 930 pounds, which would necessitate the use of the latter.

Fig. [illegible]

The guard-rails on this bridge will be placed according to the Lehigh Valley standard, and hence their inner face will be 4 feet $1\frac{1}{2}$ inches from the center of the track.

Elevation blocks will not be required, as the bridge is on a tangent.

72. The Web. The economic depth of the web, according to Article [illegible], Part I, will be:

$$\text{Depth} = \frac{61 \text{ ft. } 9 \text{ in.}}{[illegible] \times 61 \text{ ft. } 9 \text{ in.} + 0.543} = 72.5:$$

The depth might be taken as 72 inches, but 74 inches will be decided upon, as this will decrease the area of the flange and also will not affect the total weight to any great extent. The unit-stress for shear is 9 000 pounds per square inch (see Specifications, Articles 40 and 41).

The maximum shear in the girder occurs at the end, where it is 117 800 pounds. The area required for the web is then 117 800 ÷ 9 000 = 13.09 square inches, and the required thickness is 13.09 ÷ 74 = 0.177 inch. This latter value cannot be used, since, on account of Article 82 of the Specifications, no material less than $\frac{3}{8}$ inch can be used. The web plate will therefore be taken as 74 in. by $\frac{3}{8}$ in. in size.

Some engineers insist that the net section of the web should be considered. Consider Fig. 136, the shear being transferred to the web by the end rivets. The web will not tend to shear along the section *B–B*, in which case the rivet-holes should be subtracted; but it will shear along section *A–A*, a section which is unaffected by the rivet-holes. The web splice should come at one of the stiffeners, and will therefore be considered in Article 76.

73. The Flanges. This portion of the girder is usually built either of two angles or of two angles and one or more plates. In heavy girders where the flange areas are large, additional area is obtained by using side plates or side plates and four angles. Sometimes two channels are used in the place of side plates and angles. Fig. 137 shows the different methods of constructing the top flanges of girders. The lower flanges are usually of the same construction. Fig. 137 *b* has the web extending beyond the upper sur-

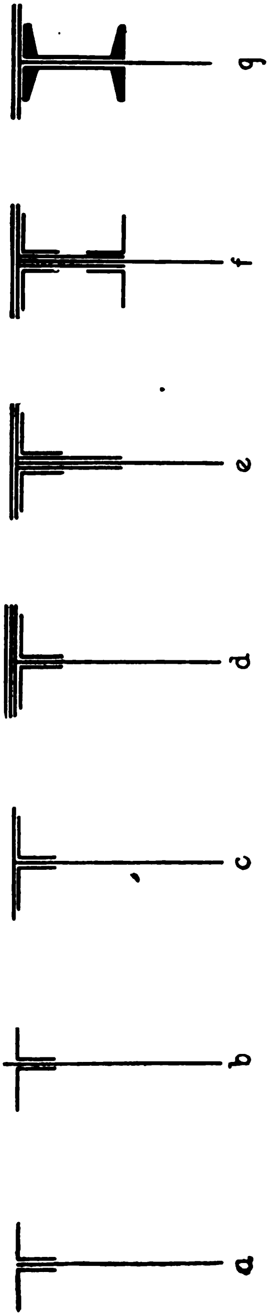


Fig. 137. Methods of Constructing Top Flanges of Girders.

faces of the upper flange angles. This is done in order that the ties may be dapped over it, and thus prevent the labor usually required for cutting holes in the lower face of the tie in order to allow for the projecting rivet-heads. Fig. 137 *g* is usually uneconomical, since the thinness of the channel web requires a great many rivets to sufficiently transmit the shear from the web to the flange, and also since the cover-plates must be very narrow.

Specifications usually state that the flanges shall have at least one-half of the total flange area in the angles, or that the angles shall be the largest that are manufactured. The largest angles are not usually employed, since their thickness is greater than three-quarters of an inch and therefore the rivet-holes must be bored, not punched. The reason for this is that the depth of the rivet-hole is too great in proportion to its diameter, and on this account the dies used for punching frequently break. Also, the punching of such thick material injures the adjacent metal, which makes it undesirable. In reality the flange area of only the short-span girders is small enough to allow the flange area to be taken up by the angles.

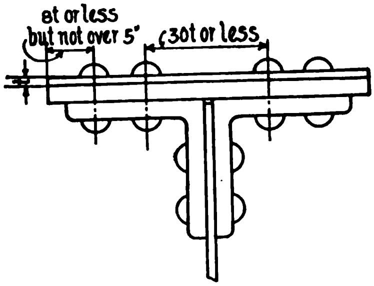


Fig. 138. Diagram Showing Relation between Thickness of Cover-Plates and Position of Rivets.

In choosing the thickness of the cover-plates, care should be taken so that the outer row of rivets will not come closer to the outer edge of the plate than eight times the thickness of the thinnest plate. In case eight times the thickness of the plate is greater than 5 inches, then 5 inches should be the limit. Also, the distance between the inner rows of rivets should not exceed thirty times the thickness of the thinnest plate. These limitations are placed by Article 77 of the Specifications, and Fig. 138 indicates their significance.

The determination of the required flange area depends upon the distance between the centers of gravity of the flanges; and in order to determine this exactly, the area and composition of the flanges should be known. The above condition requires an approximate design to be made, the supposition being that the flanges consist of two angles and one or more plates as shown in Fig. 138.

The distance back to back of angles will be taken as $74 + 2 \times \frac{1}{8} = 74\frac{1}{4}$ inches. Article 74 of the Specifications requires $\frac{1}{16}$ inch; but $\frac{1}{8}$ inch is better practice, since the edges of the web plate are very liable to overrun more than $\frac{1}{16}$ inch. Some specifications require $\frac{1}{4}$ inch.

In the computation of the approximate flange area, the center of gravity of each flange will be assumed as one inch from the back of the angles. The approximate effective depth is then $74\frac{1}{4}$ less 2×1 inch, which equals $72\frac{1}{4}$ inches. The approximate stresses in the flange areas are:

$$\text{For dead load, } \frac{275\,000 \times 12}{72.25} = 45\,600 \text{ pounds.}$$

$$\text{For live load, } \frac{1\,340\,000 \times 12}{72.25} = 222\,000 \text{ pounds.}$$

The approximate flange areas are now obtained by dividing these amounts by the allowable unit-stresses for dead and live load, which are (see Specifications, p. 8, Article 31): 20 000 and 10 000 pounds per square inch respectively; and the resulting areas are:

$$\text{For dead load, } \frac{45\,600}{20\,000} = 2.28 \text{ square inches.}$$

$$\text{For live load, } \frac{222\,000}{10\,000} = 22.20 \text{ square inches.}$$

These amounts give a total of 24.48 square inches as the approximate net flange area required.

It will be assumed that one-half the total area, or $24.48 \div 2 = 12.24$ square inches, is to be taken up by angles. If 12.24 sq. in. is distributed over two angles, then $12.24 \div 2 = 6.12$ square inches is the net area for one angle. Of course it is not to be assumed that the area of the angle chosen must be exactly 6.12, but that this 6.12 square inches is the approximate area of the angle to be chosen, and the net area of the angle (see Specifications, Article 149) must not be $2\frac{1}{2}$ per cent less than this, although it may be greater.

From Steel Construction, Part I, Table VII, or from the Carnegie Handbook, p. 117, a 6 by 6 by $\frac{3}{4}$-inch angle gives a gross area of 8.44 square inches and a net area of $8.44 - 2 \times (\frac{7}{8} + \frac{1}{8}) \times \frac{3}{4} = 6.94$ square inches, $\frac{7}{8}$-inch rivets being used and so spaced that two rivets are taken out of each angle (see Specifications, Article 63, and Fig. 139). A 6 by 4 by $\frac{13}{16}$-inch angle, giving a gross area of 7.47 and a net area of 6.66 square inches, one rivet-hole being out, could have been used, but $\frac{13}{16}$ inch is too thick to punch, and therefore the above angle is chosen.

The required net area of the cover-plate is now found to be $24.48 - 2 \times 6.94 = 10.60$ square inches. Since the legs of the angles are 6 inches and the thickness of the web is $\frac{3}{8}$ inch, the outer edges of the angles are $12\frac{3}{8}$ inches apart; and since the cover-plate must extend somewhat over the edges of the angle, and the width of the cover-plate should be in the even inch, the width of the cover-plates must be at least 14 inches, as shown in Fig. 139.

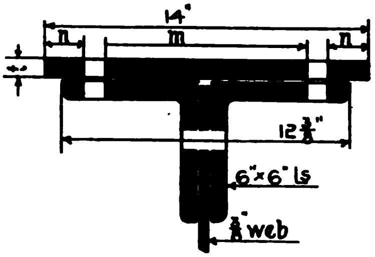


Fig. 139. Calculation of Size of Angles and Cover-Plate.

On account of the 1-inch rivet-holes to be deducted, the real or net width of the cover-plate is: $2 \times n + m = 14 - 2 \times 1 = 12$ inches. The thickness of all the cover-plates at the center is now:

$$t = \frac{10.69}{12} = 0.885 \text{ inch —say, } \tfrac{7}{8} \text{ inch.}$$

A thickness of $\frac{7}{8}$ of an inch is decided upon, for the reason that plates are rolled only to the nearest sixteenth of an inch.

The approximate section at the center has now been determined, and is:

2 Angles 6 by 6 by $\frac{3}{4}$-inch	=	13.88 sq. in. net.
Cover-plates $\frac{7}{8}$ inch thick	=	10.50 sq. in. net.
Total	=	24.38 sq. in. net.

This approximate section must now be examined, and, if it shows too great an excess or a deficiency, must be revised.

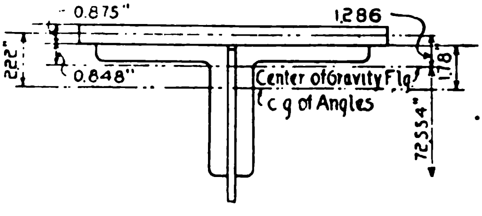


Fig. 140. Determination of Center of Gravity.

In order to determine the effective depth the distance between the centers of gravity of the flanges must first be computed, the gross areas being used. Theoretically, perhaps, the net areas should be used; but this is an unnecessary refinement, since the effect on the final result is of no practical importance.

In computing the center of gravity (see Fig. 140), the axis is aken at the center of the cover-plates, as this reduces the moment of he cover-plates to zero. The distance of the center of gravity of the ngles from their back (Carnegie Handbook, p. 117, column 6) is 1.78 inches. The distance of this center of gravity from the center f the cover-plate, is 1.78 + 0.875 ÷ 2 = 2.22 inches.

Gross area of the angles = 2 × 8.44 = 16.88 sq. in.
" " " " cover-plates = $\frac{7}{8}$ × 14 = 12.25 sq. in.
Total = 29.13 sq. in.

The center of gravity is now found to be 16.88 × 2.22 ÷ 29.13 = 1.286 inches from the center of the cover-plate, and 1.286−0.875 ÷2=0.848 inch from the back of the angle. The effective depth h_e is 74.25 − 2 × 0.848 = 72.554 inches, and the required flange areas are:

$$\frac{275\,000 \times 12}{72.554 \times 20\,000} = 2.272 \text{ sq. in. for dead load.}$$

$$\frac{1\,340\,000 \times 12}{72.554 \times 10\,000} = \underline{22.200} \text{ sq. in. for live load.}$$

$$\text{Total} = 24.472 \text{ sq. in.}$$

The values of the moments, as taken from the curves, must be multiplied by 12 in order to reduce them to pound-inches.

A total of 24.86 square inches is given by the section approximately designed, and the difference between that and the section as above determined is: (24.472 − 24.38) ÷ 24.472 = 0.38 per cent, and as this is less than 2½ per cent (see Specifications, Article 149), it may be used without any further change. If there should have been a deficiency or an excess greater than 2½ per cent, then it would have been necessary to revise. In case a revision of section is necessary, the size and thickness of the angles generally remain the same as those taken in the approximate design, the thickness of the cover-plates being decreased or increased as the case may be.

The total thickness of the cover-plates, $\frac{7}{8}$ inch, is too thick to be punched. In such cases as this, the section is made up of two or more plates whose total thickness is equal to that required. If plates of more than one thickness are decided upon, then their thickness should decrease from the flange angles outward. For the case in hand, one plate $\frac{3}{8}$ inch thick and one plate $\frac{1}{2}$ inch thick will be decided upon. The flange section at the center as finally designed is:

SHAPE	NET SECTION	GROSS SECTION
2 Angles 6 by 6 by ¾ in.	13.88 sq. in.	16.88 sq. in.
1 Cover-plate 14 by ⅜ in.	4.50 "	5.25 "
1 Cover-plate 14 by ½ in.	6.00 "	7.00 "
Total	24.38 "	29.13 "

The above is the section required at the center of the girder; for any other point it will be less, decreasing toward the end, where it will be zero. Evidently, then, the cover-plates will not be required to extend the entire length. The following analysis will determine where they should be stopped. If the load were uniform, the moment curve would be a parabola. Although under wheel loading the curve of moments is not a parabola, yet it is sufficient for practical purposes to consider it as such. The curve of flange areas, like that of moments, is to be considered a parabola (see Fig. 141).

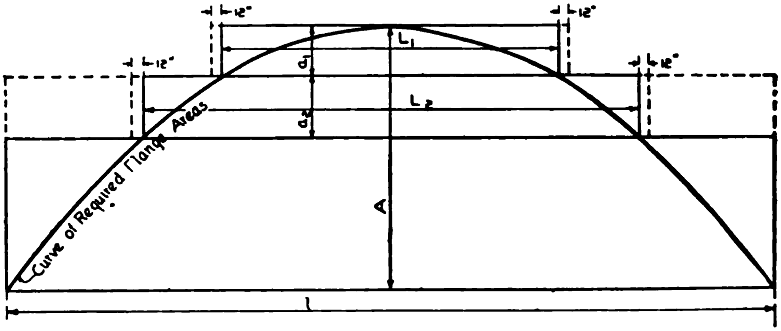


Fig. 141. Diagram Showing Curve of Required Flange Areas.

Let a_1 = Net area, in square inches, of the outer cover-plate;
a_2 = Net area, in square inches, of the next cover-plate;
a_3, etc. = Net areas of the other cover-plates;
A = Net area of all the cover-plates and the flange angle.

Then, from the properties of the parabola,

$$L = l\sqrt{\frac{\Sigma a}{A}},$$

where L = Length of cover-plate in question;
l = Length of span, center to center;
a = Net area of that cover-plate and all above it; and
A = Total net area of the flange, ⅛ of the gross area of the web not being considered in this quantity.

The lengths of the cover-plates for the section above designed (see Fig. 141) are:

$$L_1 = 61.75\sqrt{\frac{4.50}{24.38}} = 26.45 \text{ feet.}$$

$$L_2 = 61.75\sqrt{\frac{4.50 + 6.00}{24.38}} = 40.00 \text{ feet.}$$

One foot is usually added on each end of the cover-plate as theoretically determined above. The results are also usually rounded off to the nearest half-foot. This is done in order to allow a safe margin because of the fact that the curve of flange areas is not a true parabola. The final measurements of the cover-plates are:

14 in. by ⅜ in. by 28 ft. 6 in. long.
14 in. by ½ in. by 42 ft. 6 in. long.

In most cases the cover-plate next to the angle on the top flange only is made to extend the entire length of the girder. Although this is not required for flange area, it is done in order to provide additional stiffness to the flange angles toward the ends of the span, and to prevent the action of the elements from deteriorating the angles and the web by attacking the joint at the top (see broken lines, Fig. 141, for length of first cover-plate extended).

EXAMPLES FOR PRACTICE

1. The dead-load moment equals 469 000 pound-inches; and the live-load moment, 4 522 000 pound-inches. Design a flange section entirely of angles, if the distance back to back of angles is 45¼ inches.

2. The dead-load moment is 3 340 000 pound-inches, and the live-load moment, 21 235 000 pound-inches. Design a flange section using 6 by 6-inch angles and three 14-inch cover-plates, the distance back to back of flange angles being 78¼ inches.

3. In each of the above cases, design the flange section considering that ⅛ of the web area is taken as effective flange area. (For demonstration of the methods to be employed in the solution of this problem, see the succeeding text.)

While the section of a plate-girder is composite—that is, it consists of certain shapes joined together, and is not one solid piece—nevertheless these shapes are joined so securely that the section may be considered as a solid one and its moment of resistance computed accordingly. Let Fig. 142 be considered.

The moment of resistance of the section is:

$$M = \frac{S\left(A \times \frac{h^2_e}{2^2} + A \times \frac{h^2_e}{2^2} + \frac{th^3}{12}\right)}{\frac{h}{2}},$$

in the derivation of which the moment of inertia of the flange about its own neutral axis is considered as zero, and A equals the net area of one flange. Now, as the values of h_e and h seldom differ by more than one inch, for all practical purposes they may be considered as equal. The above expression then reduces to:

$$M = S \times h\left(A + \frac{th}{6}\right)$$

$$= S \times h \text{ (net area of flange + one-sixth gross area of web)}$$

Since the rivet-holes decrease the moment of resistance of the web, one-sixth of the gross area cannot be considered, as is theoretically indicated in the above formula. It is common practice to take one-eighth, instead of one-sixth, of the gross web area. Substituting this value in the above equation, and transposing, there results:

$$\text{Area of flange} + \tfrac{1}{8} \text{ gross web area} = \frac{M}{Sh}.$$

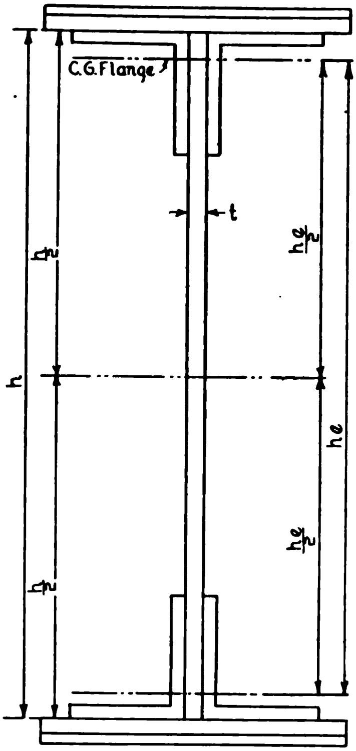
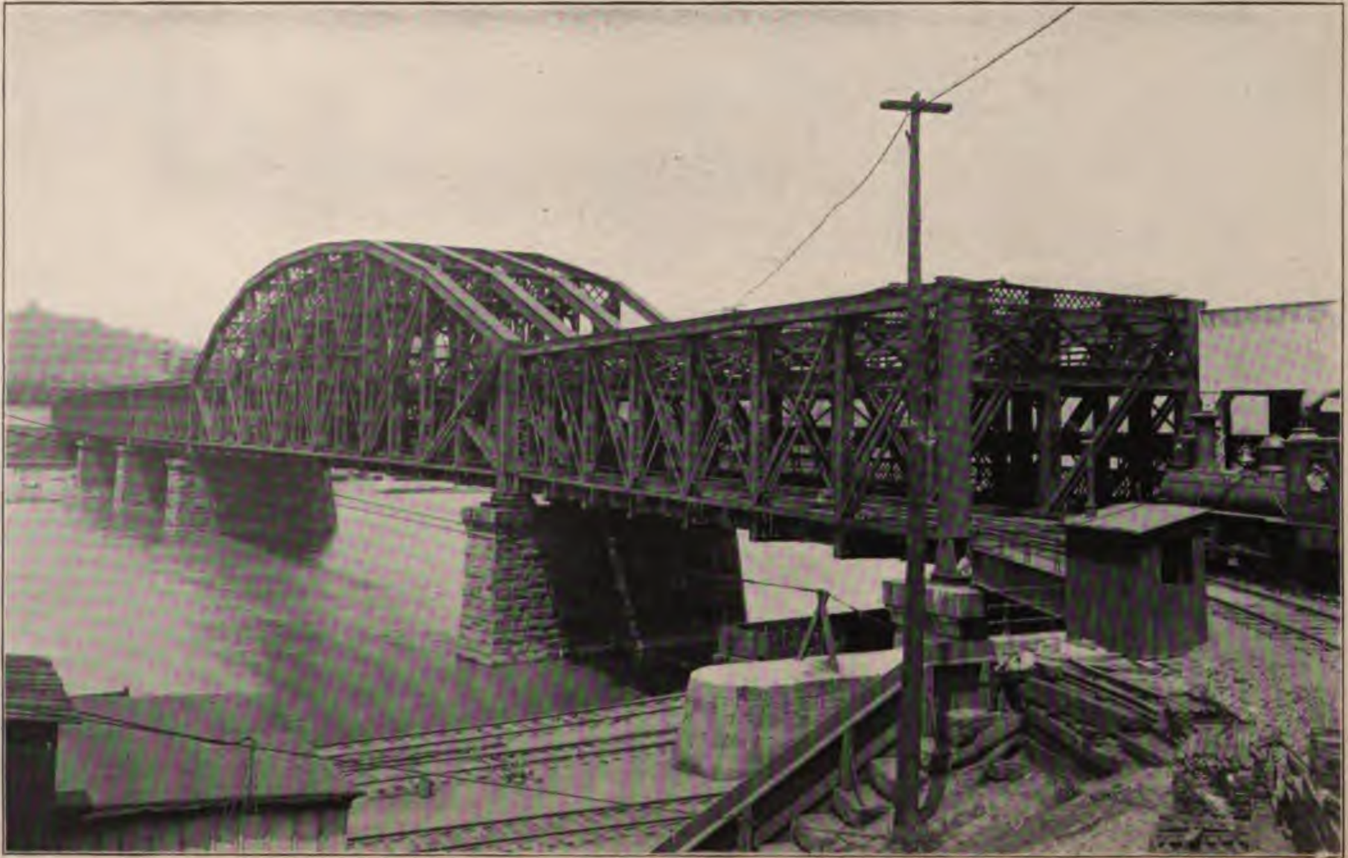


Fig. 142. Section of Plate-Girder.

The flange section will now be designed for the moments previously given, considering $\frac{1}{8}$ of the gross web area as efficient in withstanding the moment.

The gross area of the web is $74 \times \frac{3}{8} = 27.75$ square inches; and $\frac{1}{8}$ of this is 3.47 square-inches. The total approximate amount of flange area required is, as in the first case, 24.48 square inches.

According to the above formula, $\frac{1}{8}$ of the web area, or 3.47 square inches, may be considered as flange area, and therefore $24.48 - 3.47 = 21.01$ square inches, is the approximate area of the angles and cover-plates of the flange. The approximate area of one angle is then $21.01 \div (2 \times 2) = 5.25$ square inches. A 6 by 6 by $\frac{9}{16}$-inch angle gives the gross area of 6.43 square inches and, two rivet-holes being deducted, a net area of 5.305 square inches (see "Steel Construction," Part I, Table VIII, or Carnegie Handbook, p. 117). As this is quite close to the approximate area determined above, this angle will be taken. The ap-



BRIDGE OVER ALLEGHENY RIVER, PITTSBURG, PENNSYLVANIA

On line of Pittsburg, Fort Wayne & Chicago Railroad. Built in 1901. Consists of the following four-track double-deck through pin-connected spans: One of 157 ft., one of 333 ft., and three of 160 ft. each, with a double-track viaduct at one end, 816 ft. long.

Courtesy of American Bridge Company.

proximate area of the cover-plates is 21.01 − 2 × 5.305 = 10.40 square inches. As before, the gross width of the cover-plate will be taken as 14 inches. The thickness is then $\frac{10.40}{14 - 2(\frac{7}{8} + \frac{1}{8})}$ = 0.867 inch—say $\frac{7}{8}$ inch.

The gross area of the angles being 12.86 square inches, and that of the cover-plates 12.25 square inches, the center of gravity of the section is found, by a method similar to that previously employed, to be 1.10 inches from the center of the cover-plate, or 1.10 − 0.438 = 0.662 inch from the back of the flange angles. This makes the effective depth 72.93 inches.

For this section, the true live-load flange stress is (1 340 000 × 12) ÷ 72.93 = 221 000 pounds, and the actual dead-load flange stress is (275 000 × 12) ÷ 72.93 = 45 400 pounds. The actual areas required for the live and dead load are 22.10 and 2.27 square inches, which are obtained by dividing the above flange stresses by 10 000 and 20 000 pounds, respectively. The total required area is the sum of the two areas above, and is equal to 24.37 square inches. The total area required in the flange angles and cover-plates is therefore 24.37, less $\frac{1}{8}$ the gross area of the web, 3.47, which leaves 20.90 square inches. The same angles as decided upon before will be used. This gives a required area for the cover-plates of 20.90 − 10.61 10.29 square inches. The required thickness is then 10.29 : (14 − 2) = 0.857—say $\frac{7}{8}$ inch. The following section of the flange will therefore be decided upon:

SHAPE.	NET SECTION	GROSS SECTION
2 Angles 6 in. by 6 in. by $\frac{9}{16}$ in.	10.61 sq. in.	12.86 sq. in.
1 Cover-plate 14 in. by $\frac{3}{8}$ in.	4.50 "	5.25 "
1 Cover-plate 14 in. by $\frac{1}{2}$ in.	6.00 "	7.00 "
Total	21.11 "	25.11 "

As the total net area above is within $2\frac{1}{2}$ per cent of the required net area, that section will be taken (see Specifications, Article 149). Note that in this case, the thickness of the cover-plates in the final design is the same as that determined in the preliminary design. Also note that the total net area is about 4 square inches, or 20 per cent, less than in the flange as first designed, in which case none of the area of the web was considered as withstanding the bending moment.

The ½-inch cover-plate on the top flange will extend the entire length of the grider, and is therefore 62 feet 9 inches long. The lengths of the other cover-plates are:

$$\text{For ½-inch plate at the bottom, } L = 61.75\sqrt{\frac{10.50}{21.11}} = 43.5 \text{ feet.}$$

$$\text{For each ⅜-inch plate, } L = 61.75\sqrt{\frac{4.50}{21.11}} = 28.5 \text{ feet.}$$

One foot should be added to each of the above lengths at each end, thus making the total lengths 45 feet 6 inches and 30 feet 6 inches, respectively.

EXAMPLES FOR PRACTICE

1. If the span is 63 feet center to center, compute the length of the cover-plate. The section consists of two angles 6 by 6 by ⅜ in.; one cover-plate 14 by ½ in.; and one cover-plate 14 by 7/16 in.; two rivet-holes being taken out of each angle and each cover-plate.

2. If the span is 87 ft. 9 in. center to center, compute the length of the cover-plate if the flange consists of two angles 6 by 6 by ¾ in., and four cover-plates 16 by 9/16 in., two rivet-holes being taken out of each angle and each cover-plate.

In determining the area of plates, the tables in the Carnegie Handbook, pp. 245 to 250, are convenient. In order to obtain plates whose widths are greater than 12¾ inches, see the note in the right-hand column on page 250, Carnegie Handbook. For another presentation of the above subject-matter, see "Steel Construction," Part IV. pp. 252, 254, and 261.

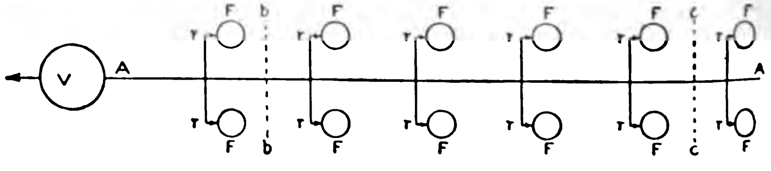


Fig. 143. Diagram Illustrating Transference of Shear from Web to Flanges by Rivets.

The spacing of the rivets in the flanges is a matter of considerable importance; the shear is transferred from the web to the flanges, where it becomes flange stress. This is done by the rivets, each rivet taking as much flange stress as is allowed by the Specifications. The conditions are similar to those shown in Fig. 143, where V represents an object exerting a pull on a long, thin plate $A-A$ which has, at various points along this length, small objects F attached to it by means of pegs $r-r$. These small objects F hold the plate $A-A$ in equilibrium. Here V represents the shear which tends to cause the movement; $A-A$, the web; $r-r$, the rivets; and $2F$ the amount of flange stress taken by each rivet.

At section $c-c$ the total amount in the web to be transformed is $2F$; at section $b-b$ it is $10F$. From this it is seen that enough rivets $r-r$ must be put in between the sections $b-b$ and $c-c$ to take up $10F - 2F = 8F$; hence it is proved that the rivets between any two sections of the flange take up the difference in flange stress between those two sections.

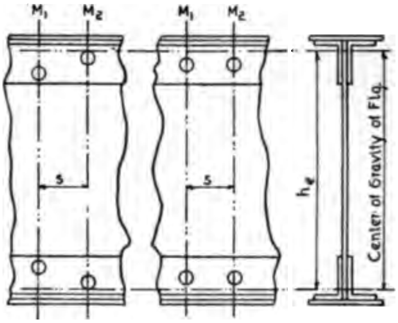


Fig. 144. Determination of Rivet Spacing.

The discussion just given will be the means of giving us the number of rivets required between any two sections; but it does not give us the rivet spacing between these two sections. In order to determine the rivet spacing at any particular point, the following analysis is presented (see Fig. 144).

Let M_1 = Moment at one section;
M_2 = Moment at another section nearer center of girder than the section where M_1 occurs;
V = Shear at section where M_1 occurs;
v = Amount of flange stress one rivet can transfer; or it is the stress on one rivet;
s = Distance between the two sections;
n = Number of rivets between the two sections.

Then,

$\frac{M_1}{h_e}$ = Flange stress due to moment M_1;

$\frac{M_2}{h_e}$ = Flange stress due to moment M_2;

$\frac{M_2}{h_e} - \frac{M_1}{h_e}$ = Difference of flange stress between the two sections;

$\left(\frac{M_2}{h_e} - \frac{M_1}{h_e}\right) \div v = n$, Number of rivets required in space s. . .(1)

If the above sections be taken close enough together so that the number of rivets required will be 1 (that is, n = 1), then V can be considered as constant between the two sections, and then the moment $M_2 = M_1 + Vp$ (see Article 44, Part I). Substituting in Equation 1, above, there results:

$$\left(\frac{M_1 + Vp}{h_e} - \frac{M_1}{h_e}\right) \div r = 1,$$

from which,

$$p = \frac{rh_e}{V},$$

which is the formula for the rivet spacing in the vertical parts of the flanges of any girder, providing the flange is not subjected to localized loading. It is to be used for the rivet spacing in both the top and bottom flanges of through girders, but not in the top flanges of deck plate-girders for railroad service. It is to be used, however, in the bottom flanges of deck plate-girders for railroad service. The distance h_e is not ordinarily used, the distance between rivet lines being used instead (see Fig. 145). The rivet spacing in the cover-plates and horizontal legs of the angles is made to stagger with that in the vertical legs, [illegible] so that the staggering is with every other rivet in the vertical flange. The term stagger signifies that the rivets in the top flange are not placed opposite the rivets in the vertical legs of the flange angles, so that in case there are two lines of rivets in the vertical legs of the angles, a rivet near the outer edge of the cover-plate is placed at the same section where a rivet occurs near the lower edge of the vertical legs of the angle, and *vice versâ*.

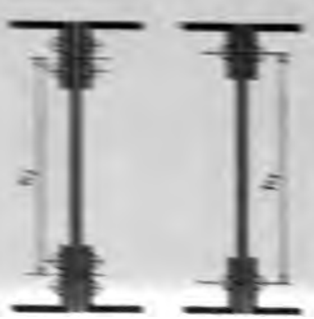


Fig. 145. [illegible]

EXAMPLES FOR PRACTICE

1. Determine the rivet spacing at a section where the shear is 147 200 pounds, the value of one rivet 4 920 pounds, and the effective depth of the section 84¼ inches.

Ans. 2.82 inches.

2. Determine the stress on a rivet at a section where the shear is 299 400 pounds, the spacing 2½ inches, and the effective depth of the girder 84¼ inches.

Ans. 8 870 pounds.

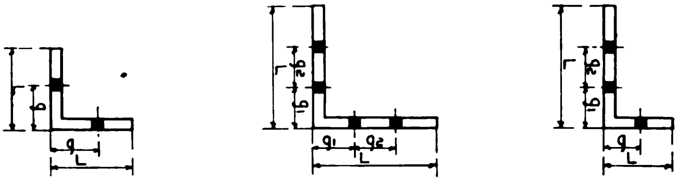
The rivet spacing is usually determined at the tenth-points, and a curve is plotted with the spacing as ordinates and the tenth-points as abscissæ. The rivet spacing at any intermediate point can be determined from this curve. When one-eighth the gross section of the web is considered as flange area, then only that proportion of the shear which is transferred to flanges is to be considered in computing the rivet spacing, on account of the fact that some of the shear is transferred directly to the bending moment in the web.

In order to determine the distance between rivet lines, the *gauge*, or distance out from the back of the angles to the place where the rivets must be placed, must be known for different lengths of legs. Table XXI gives the standard gauges, and also the diameter of the largest rivet or bolt which is allowed to be used in any sized leg. No gauges should be punched otherwise unless your large experience or instructions from one higher in authority demand it, and this should be so seldom that indeed it might be said never to be necessary.

TABLE XXI

Standard Gauges for Angles

(All dimensions given in inches)



L	*g*	Maximum Rivet or Bolt	*L*	*g*	Maximum Rivet or Bolt	*L*	*g*	Maximum Rivet or Bolt
8	4½	⅞	3½	2	⅞	2	1⅛	½
7	4	⅞	3	1¾	⅞	1¾	1	½
6	3½	⅞	2¾	1⅝	¾	1½	⅞	⅜
5	3	⅞	2½	1⅜	⅝	1¼	¾	⅜
4	2¼	⅞	2¼	1¼	⅝	1	9/16	¼

L	*g*₁	*g*₂	*L*	*g*₁	*g*₂
8	3	3	6*	2½	2¼
7	2½	3	5	2	1¾
6	2¼	2½			

*When thickness is ¾ inch or over.

The distance between rivet lines for the girder being designed (see Fig. 146), is, in the first case:

$$h_r = h - \left(2g_1 + \frac{2g_1}{2}\right)$$
$$= 74.25 - (2 \times 2\tfrac{1}{2} + 2\tfrac{1}{4})$$
$$= 67.00 \text{ inches.}$$

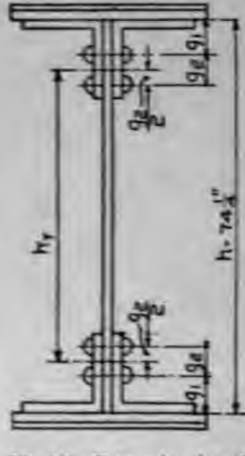


Fig. 146. Determination of Distance between Rivet Lines.

In the second case, where $\frac{1}{8}$ of the web is considered, the above distance is 74.25 − $(2 \times 2\frac{1}{4} + 2\frac{1}{2}) = 67.25$ inches. The computations and the rivet spaces at the tenth-point, and at the ends of the cover-plates in the bottom chord of the plate-girder, are shown for each case in Table XXII. The value of v is the value of a $\frac{7}{8}$-inch rivet in bearing in a $\frac{3}{8}$-inch web (see Specifications, Article 40, and Carnegie Handbook, p. 195, second table). This value is 4 920 pounds.

In the first column, 7.98− indicates that the end of the cover-plate next to the flange is 7.98 feet from the end of the girder, and that this section is taken just to one side of that point, the side being that nearest the end of the girder. In a similar manner, 7.98+ indicates that the section is taken to that side of the point which is nearest the center of the girder. A like interpretation should be placed on 15.55− and 15.55−, the point under consideration in this case being the end of the outer or top cover-plate.

In the fifth column, values are given which indicate that portion of the shear which is transferred to the flanges. For example,

$$\frac{97\ 700}{(14.08 \div 10.61)} = 74\ 700,$$

and the difference between 97 700 and 74 700 represents that portion of the shear which is taken up directly by the web in the form of bending moment. An inspection of the headings of the third and fourth columns will tend to make this matter clearer.

Where there is local loading, as in the top flange, the rivets, in addition to the stress caused by the transferring of web stresses, are stressed by the vertical action of the angles being pressed downward by the ties and the consequent upward pressure of the web. Accord-

TABLE XXII

Rivet Spacing in Bottom Flange

Flange Taking All the Moment

SECTION	TOTAL SHEAR (Pounds)	h_r (Inches)	v (Pounds)	RIVET SPACING (Inches)	REMARKS
0	117 800	67	4 290	2.80	
1	97 700	67	4 290	3.38	
2	79 300	67	4 290	4.16	
3	61 300	67	4 290	5.38	
4	44 200	67	4 290	7.46	See Specifications, Art.54
5	28 600	67	4 290	11.52	

One-Eighth of Web Area Considered

h_r = 67.25 inches; v = 4 920 pounds

SECTION	TOTAL SHEAR (Pounds)	NET FLANGE AREA PLUS ⅛ WEB AREA (Sq. Inches)	NET FLANGE AREA (Sq. Inches)	REDUCED SHEAR (Pounds)	RIVET SPACING (Inches)
0	117 800	14.08	10.61	88 800	3.75
1	97 700	14.08	10.61	74 700	4.45
7.98−	90 000	14.08	10.61	67 900	4.88
7.98+	90 000	20.08	16.61	74 800	4.42
2	79 300	20.08	16.61	65 800	5.02
15.55−	67 500	20.08	16.61	56 000	5.81
15.55+	67 500	24.58	21.11	58 100	5.69
3	61 300	24.58	21.11	52 700	6.28
4	44 200	24.58	21.11	38 000	8.71
5	28 600	24.58	21.11	24 600	13.42

ing to Article 15 of the Specifications, the weight of one driver is distributed over three ties (see Fig. 147).

Let $\frac{W}{l}$, = w, the load per linear inch caused by one wheel W, which load is assumed to be uniformly distributed over the distance l;

$ws = v_1$, the vertical load or stress that comes on one rivet in the space s;

$v = \frac{Vs}{h_r}$, the stress on a rivet due to the distribution of flange stresses when s is a space, and V the shear at that point.

When these two stresses act on the rivet, the maximum stress will be v_u, the ultimate amount that it is allowed to carry, and this will act as shown in Fig. 147: Then,

$$v_u = \sqrt{v^2 + v_1^2}$$

$$= \sqrt{\left(\frac{V_s}{h_r}\right)^2 + (ws)^2}$$

$$= s\sqrt{\left(\frac{V}{h_r}\right)^2 + w^2},$$

from which,

$$s = \frac{v_u}{\sqrt{\left(\frac{V}{h_r}\right)^2 + w^2}},$$

which gives the spacing at any point in the girder flange under localized loading. Note that if w equals zero—that is, if there is no localized loading—there results:

$$s = \frac{v_u}{\sqrt{\left(\frac{V}{h_r}\right)^2 + 0}}$$

$$= \frac{v_u\, h_r}{V},$$

which is the same as previously deduced for flanges without localized loadings.

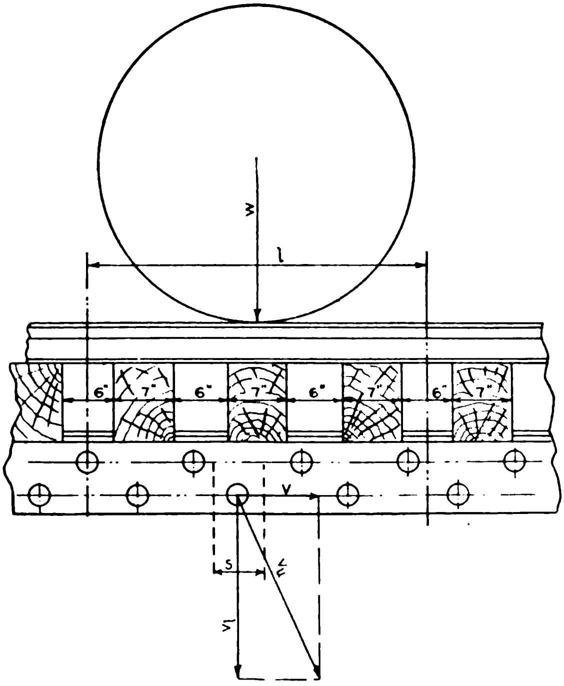


Fig 147. Rivet Spacing Determined by Stresses Distributed under Localized Loading.

The rivet spacing for the top flange of the girder which is being designed is given in Table XXIII. Here $W = 20\,000$; $l = (3 \times 7 + 3 \times 6) = 39$ inches; $w = \frac{20\,000}{39} = 513$; $h_r = 67$ inches; and $v_w = 4\,920$ pounds, which is the bearing of a $\frac{7}{8}$-inch rivet in the $\frac{3}{8}$-inch web. The top cover-plate is run the entire length of the span.

TABLE XXIII

Rivet Spacing in Top Flange

Flange Taking All the Moment

SECTION	w^2	TOTAL SHEAR (Pounds)	$\left(\frac{V}{h_r}\right)^2$	$\sqrt{\left(\frac{V}{h_r}\right)^2 + w^2}$	RIVET SPACE (Inches)
0	262 600	117 800	3 080 000	1 825	2.70
1	"	97 700	2 140 000	1 550	3.17
2	"	79 300	1 390 000	1 285	3.83
3	"	61 300	840 000	1 050	4.68
4	"	44 200	435 000	835	5.88
5	"	28 600	181 500	660	7.45

One-Eighth of Web Area Considered

$w = 513$; $h_r = 67\frac{1}{4}$ inches; $v_u = 4\,920$ pounds.

SECTION	w^2	REDUCED SHEAR (Pounds)	$\left(\frac{V}{h_r}\right)^2$	$\sqrt{\left(\frac{V}{h_r}\right)^2 + w^2}$	RIVET SPACE (Inches)
0	262 600	97 600	2 100 000	1 538	3.20
1	262 600	81 000	1 450 000	1 315	3.74
2	262 600	65 800	985 000	1 100	4.50
15.55 –	262 600	56 000	695 000	980	5.02
15.55 +	262 600	58 100	765 000	1 014	4.85
3	262 600	52 700	616 000	938	5.24
4	262 600	38 000	320 000	763	6.41
5	262 600	24 600	134 000	629	7.82

The points other than the tenth-points referred to in the first column are for sections taken just to the left and right of the top cover-plate. The values of the reduced shears given in the third column are obtained as has been previously explained. Although the rivet spacing in the lower flange is considerably greater than that in the upper flange, and accordingly a smaller number of rivets would be required, yet the spacing in the lower flange is made the same as that in the upper. Convenience in the preparing of plans and facility in manufacture make this action economical. Theoretical spacing greater than 6 inches should be dealt with according to Article 54 of the Specifications.

The values of the rivet spacing given in Tables XXII and XXIII are plotted in Fig. 148. Note that the effect of the localized loading is to decrease the rivet spacing, and also note that the effect increases from the ends toward the center.

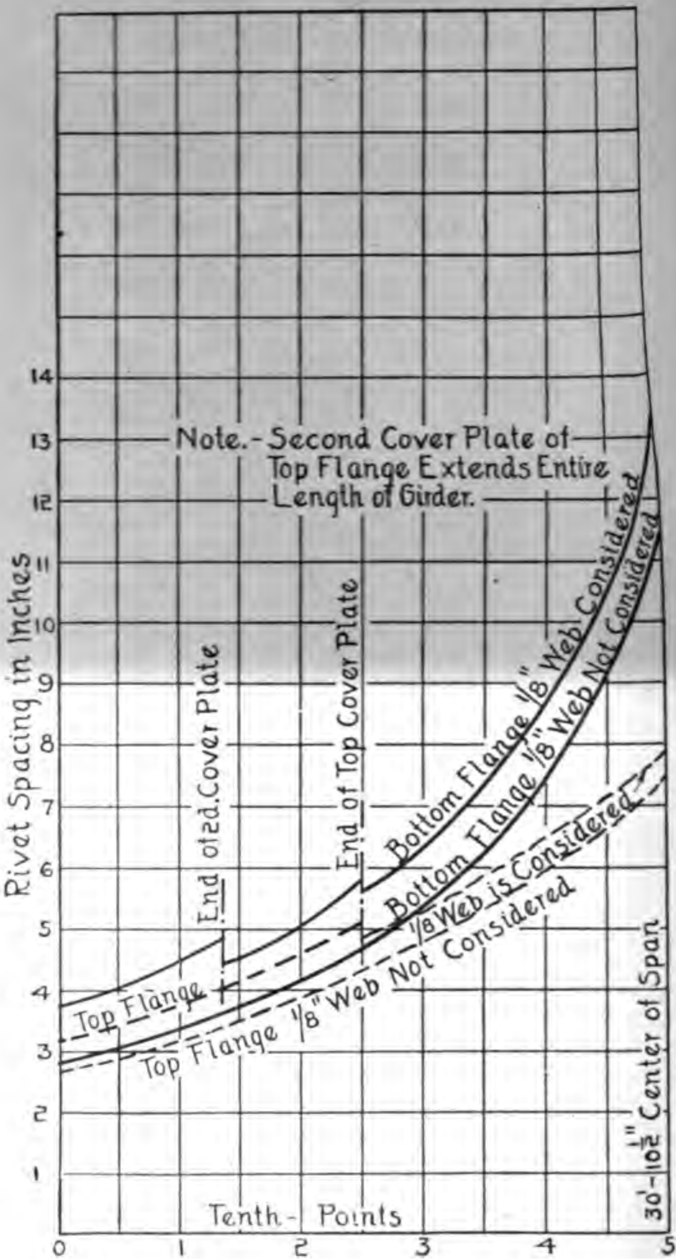


Fig. 148. Plotted Values of Rivet Spacing Given in Tables XXII and XXIII.

diagonal bracings (see Plate II), and are placed at [illegible] according to the judgment of the engineer. [illegible] that their number should be about as indicated below:

Span of Girder	Number of Cross-Frames
0 to 20 feet	-
20 to 35 "	[illegible]
35 to 70 "	[illegible]
70 to 85 "	[illegible]
85 to 110 "	[illegible]

The above is not intended to serve as a hard and fast rule. Variations from the limits given are to be made as the case demands. In all cases they are put at the panel points of the bracing, [illegible] bottom parts acting as sub-verticals in the lateral system. [illegible] cross-frames should divide the span into equal parts if possible. In cases where that is not possible, the shortest distances should come near the ends of the spans.

If the panel points are to be located at the stiffeners the number of panels is a function of the depth of the girder (see Specifications, Articles 47 and 48). In this case the number of panels is given by:

$$N = \frac{\text{Span in inches}}{\text{Depth of girder in inches}},$$

no fraction being considered. As an example, let it be required to determine the number of panels in a girder 85 feet [illegible] of bearings, the depth being 90¼ inches back to back of angles. Then

$$N = \frac{85 \times 12}{90.25} = 11 \text{ [illegible] } 11 \text{ panels}.$$

Each panel will then be 92.8 inches long. This, according to Article 47 of the Specifications, being greater than 5 feet, would not be allowed as a space between two stiffeners, but one stiffener can be placed in between, and then the panel points will come at every other stiffener. The cross-frames should be five in number.

The arrangement of panels and cross-frames is shown in Fig. 149. Here the cross-frames are marked *C. F.*, and the broken lines represent the lower lateral system.

In case an even number of panels were desired, then ten would be the number chosen and the general arrangement would be as shown in Fig. 150. The length of a panel would be 85 × 12 ÷

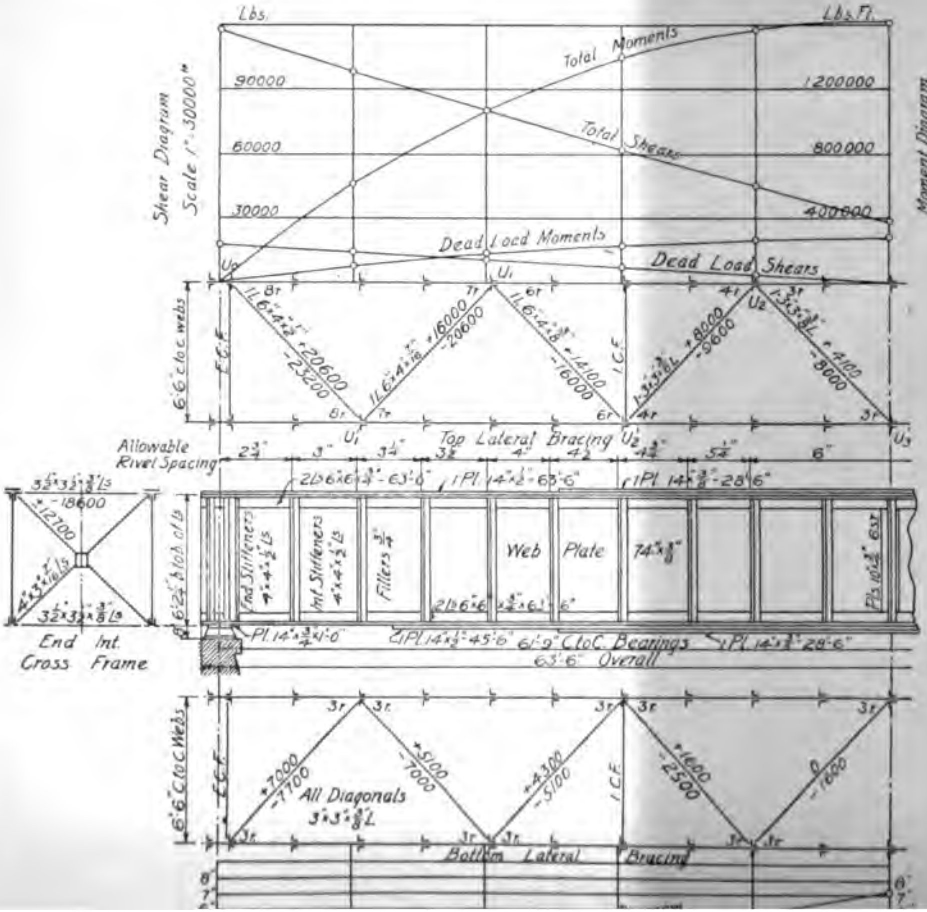


PLATE II.

Dead Load W = L (124 + 10L) = 47200 + Track = 72300#
Live Load = E 40
Specifications Cooper's 1901
All Rivets ⅞" Diameter

Maximum Shears
Dead Load = 17800
Live Load = 100000
Total = 117800
$\frac{117800}{9000}$ = 13.09 □" Net area required
74" × ⅜" = 27.76 □" Gross area of web
$\frac{117800}{4920}$ = 25 Rivets

Maximum Moment in Lb. Ft.
Dead Load = 275000
Live Load = 1340000
Total = 1615000

$\frac{275000 \times 12}{72.554}$ = 45400 D.L. Flange Stress
$\frac{1340000 \times 12}{72.554}$ = 221500 L.L. Flange Stress
$\frac{45400}{20000}$ = 2.27 □"
$\frac{221500}{10000}$ = 21.20 □"
24.47 □" Net area req. in flange

2 Ls 6"×6"×¾" = 13.88 □" Net.
1 C. Pl. 14"×½" = 6.00 □" Net.
1 C. Pl. 14"×⅜" = 4.50 □" Net.
Total = 24.38 □" Net.

L.V.R.R. BRIDGE No. 358A
SINGLE TRK. DECK GIRDER
Over Raritan R., Blake Co. N.J.
61'-9" C. to C. Bearings
CAPITOL BRIDGE CO.
Pekin Plant

diagonal bracings (see Plate II), and are placed at certain intervals according to the judgment of the engineer. Good practice demands that their number should be about as indicated below:

Span of Girder	Number of Cross-Frames
0 to 20 feet	2
20 to 35 "	3
35 to 70 "	4
70 to 85 "	5
85 to 110 "	6

The above is not intended to serve as a hard and fixed rule. Variations from the limits given are to be made as the case demands. In all cases they are put at the panel points of the bracing, the top and bottom parts acting as sub-verticals in the lateral system. Also, the cross-frames should divide the span into equal parts if possible. In cases where that is not possible, the shortest divisions should come near the ends of the spans.

If the panel points are to be located at the stiffeners, the number of panels is a function of the depth of the girder (see Specifications, Articles 47 and 48). In this case the number of panels is given by:

$$N = \frac{\text{Span in inches.}}{\text{Depth of girder in inches}},$$

no fraction being considered. As an example, let it be required to determine the number of panels in a girder 85 feet center to center of bearings, the depth being $90\frac{1}{4}$ inches back to back of angles. Then,

$$N = \frac{85 \times 12}{90.25} = 11.3\text{, or, say, } 11 \text{ panels.}$$

Each panel will then be 92.8 inches long. This, according to Article 47 of the Specifications, being greater than 5 feet, would not be allowed as a space between two stiffeners; but one stiffener can be placed in between, and then the panel points will come at every other stiffener. The cross-frames should be five in number.

The arrangement of panels and cross-frames is shown in Fig. 149. Here the cross-frames are marked *C. F.*, and the broken lines represent the lower lateral system.

In case an even number of panels were desired, then ten would be the number chosen and the general arrangement would be as shown in Fig. 150. The length of a panel would be 85 × 12 ÷

10 = 102 inches, or 8 feet 6 inches, which would allow of one stiffener in between and still keep the stiffener spacing within the limit of 5 feet.

The cross-frames at the ends of the span are designated as *end cross-frames*, and those in between are designated as *intermediate cross-frames*.

In case the spacing of the stiffeners is not required to be such as to coincide with the panel points of the lateral bracing, the panel length will depend upon the spacing of the girders, being equal to or greater than the spacing in order to keep the angle which the diagonals make with the girder less than 45 degrees. In this case,

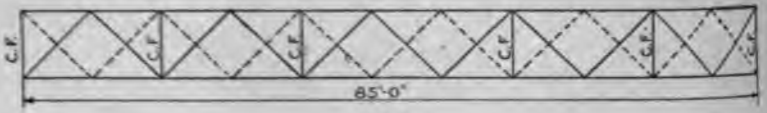


Fig. 149.

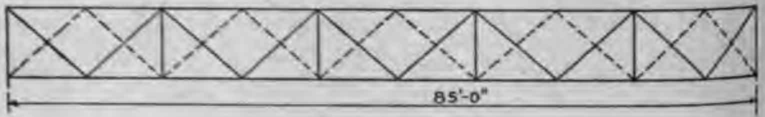


Fig. 150.

Arrangements of Panels and Cross-Frames.

$$N = \frac{\text{Span in feet}}{\text{Distance center to center of girders in feet}}.$$

For the girder considered on page 174, the number of panels would be $\frac{85}{7.5} = 11.3$—or, say, 11 panels—if odd numbers were to be used, and 12 if even numbers were to be desired.

For the case in hand, the panel points of the bracing will be taken at the stiffeners, and an even number of panels will be used. Then,

$$N = \frac{61.75 \times 12}{74.25} = 9.98 \text{ (say 10)}.$$

The arrangement of the panels and cross-frames, and also the maximum stresses in the diagonals, are shown in Plate II, the stresses being determined according to Article 50, Part I, "Bridge Analysis," and Article 24 of the Specifications. All the wind is taken as acting on one side of the bridge; and no overturning effect, either on the girder or on the train, is considered. Also, note that the wind stresses in the flanges are not considered. Should the student determine these, he will find them too small to be considered according to Specifications, Article 39.

Before designing the lateral diagonals which consist of one or two angles, Articles 31, 33, 34, 35 (last portion), 38, 40, 63, and 83 of the Specifications should be carefully studied. The upper lateral bracing is to be designed first. Carnegie Handbook, pp. 109 to 119, is to be used.

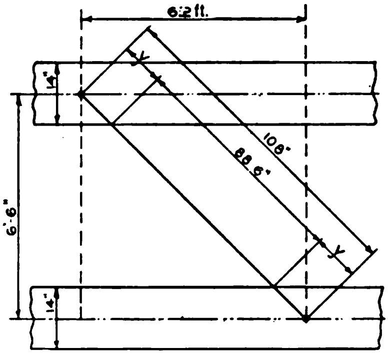


Fig. 151. Determination of Length of Diagonal in Lateral Bracing.

The member U_0U_1 must be designed for a compressive stress equal to $23.20 + 0.8 \times 20.6 = -39.68$, and a tensile stress of $20.6 + 0.8 \times 20.6 = +37.08$. The length of the diagonal measured from center to center of girders is $\sqrt{6.5^2 + 6.2^2} = 9$ feet, or 108 inches. In reality the length is not 108 inches, as the cover-plate takes off a certain amount, as shown in Fig. 151. The true length, which is to be taken as a column length in designing, is $108 - 2y$, and y is readily computed to be 9.70 inches, thus making the true length 88.6 inches. The least allowable rectangular radius of gyration is obtained from the relation that the greatest value of $\frac{l}{r} = 120$, and therefore the least value of $r = \frac{l}{120} = 0.74$.

It will be assumed that a 6 by 4 by $\frac{9}{16}$-inch angle with an area of 5.31 square inches will be sufficient. Here the length equals 88.60 inches, and the least rectangular radius of gyration is 1.14; hence,

$$P = 13\,000 - 60 \times \frac{88.60}{1.14}$$

$$= 8\,330 \text{ pounds per square inch.}$$

The required area is $\frac{39\,360}{8\,330} = 4.73$ square inches. As the angle assumed has an area of 5.31 square inches, which is considerably greater than the 4.73 square inches required, this angle cannot be used, and other assumptions must be made until the area of the angle assumed and the required area as computed are equal or very nearly so.

A 6 by 4 by $\frac{1}{2}$-inch angle with an area of 4.75 square inches will now be assumed. The length is 88.60 inches as before, and the least rectangular radius of gyration is 1.15. The unit-load $P = 8\,340$ pounds per square inch, and the required area is $\frac{39\,360}{8\,340} = 4.72$ square inches. As the area of the angle assumed and the required area as computed are very close, this sized angle will be used.

The section must now be examined for tension, and in order that both legs of the angle may be considered as effective section, both legs must be connected at the end. The area required will be $\frac{36\,760}{18\,000} = 2.04$ square inches. Considering one rivet-hole is taken out of the angle, the net area is $4.75 - 1 \times (\frac{7}{8} + \frac{1}{8}) \times \frac{1}{2} = 4.25$ square inches, which is amply sufficient.

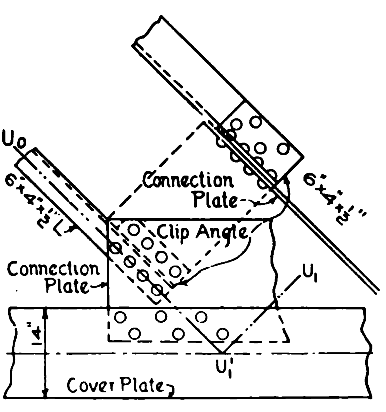


Fig. 152. Method of Connecting Angle Legs in Lateral Bracing and Cross-Frames.

If the 4-inch leg only were assumed to be connected, the gross area would be $4 \times \frac{1}{2} = 2.00$ square inches, and the net area would then be $2.00 - 1 \times (\frac{7}{8} + \frac{1}{8}) \times \frac{1}{2}$-inch $= 1.50$ square inches, which is not sufficient. If the 6-inch leg were connected, the area would be sufficient. See Fig. 152 for method of connection and rivets.

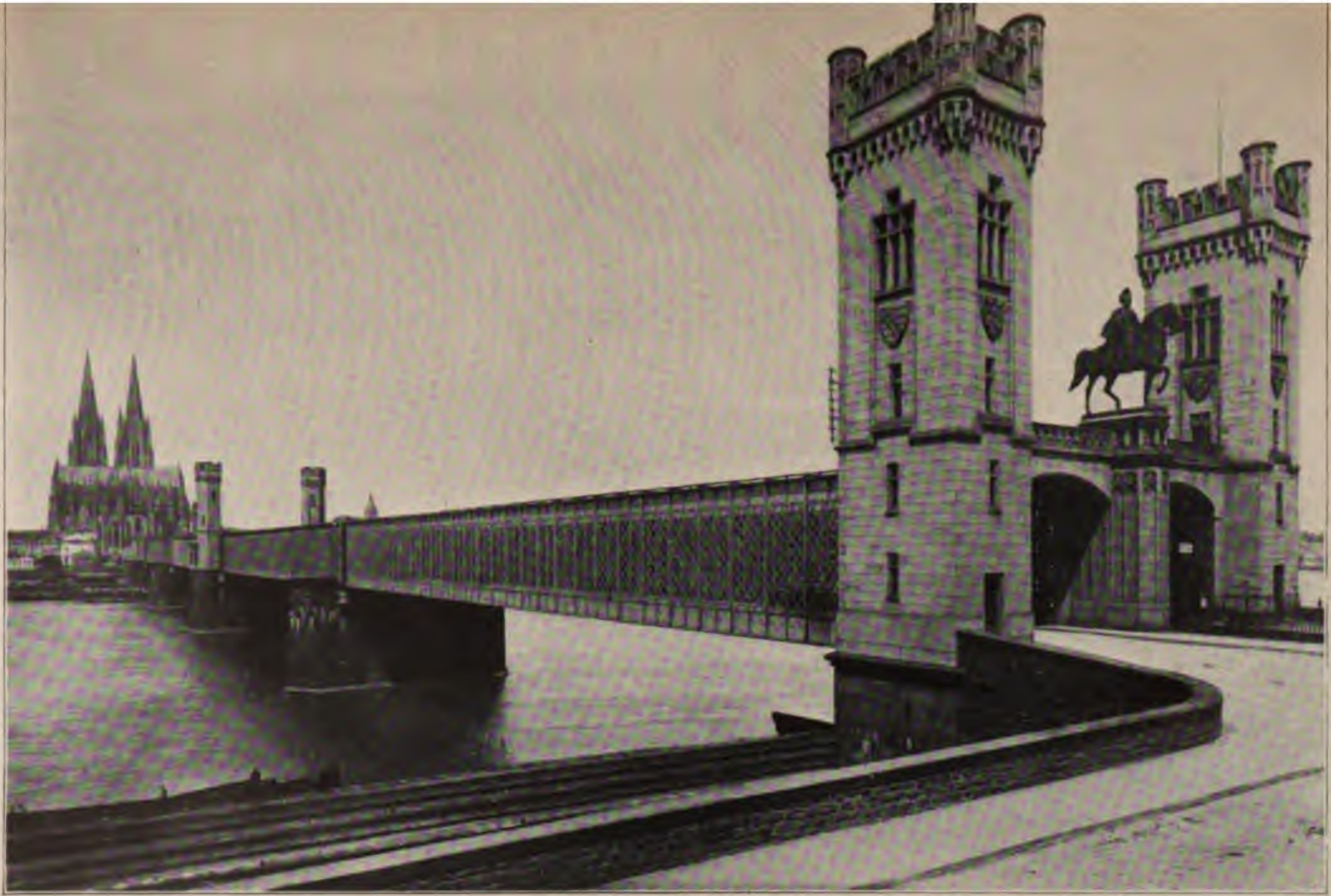
The number of rivets required (see Specifications, Article 38 and 40) is computed as follows: If the member were not subjected to both tension and compression, the number of rivets required in single shear would be:

$$n = \frac{39\,360}{(9\,000 + 50 \text{ per cent of } 9\,000) \times 0.6013}$$

$$= \frac{39\,360}{8\,100}$$

$$= 4.86 \text{ (say 5)}.$$

But according to Article 38 of the Specifications, this number must be increased 50 per cent, and accordingly $4.86 \times 1\frac{1}{2} = 7.29$ (say 8)



BRIDGE OVER THE RIVER RHINE AT COLOGNE, GERMANY

The famous Cologne Cathedral is shown at left, in background.

shop rivets are to be used. In the above formula, 0.6013 is the area of the cross-section in square inches of a $\frac{7}{8}$-inch rivet. In order that both legs should be connected, a clip angle as shown in Fig. 152 is used; and the same number of rivets must go through both legs of the clip angle, since the stress in the vertical leg of the main angle is transferred to the clip angle and from there into the connecting plate.

The above number of rivets makes the joint safe so that it will not shear off in the plane between the connection plate and the horizontal leg of the angle. The joint must also be designed so that there will be sufficient rivets in bearing to prevent them from tearing out of the connecting plate. The number required is:

$$n = \frac{39\,360}{(15\,000 + 50 \text{ per cent of } 15\,000) \times \frac{7}{8} \times \frac{3}{8}}$$

$$= \frac{39\,360}{7\,380}$$

$$= 5.34.$$

This 5.34 must be increased 50 per cent, making a total of 5.34 × 1.5 = 8.01 = say, 8 shop rivets as before.

The above rivets are shop rivets, since it is assumed that the span, being a small one, will be riveted complete in the shop and shipped to the bridge site ready to place in position without any further riveting. In case the girders are shipped separately, then the lateral bracing must be riveted up in the field; and according to last part of Article 40 of the Specifications, the rivets, being field rivets, must have the allowed unit-stresses reduced one-third, which is equivalent to having the number of shop rivets increased 50 per cent. This will make the required number of field rivets 8 × 1.5 = 12.

As a rule, the connection plates are $\frac{3}{8}$ inch thick, seldom more. Also, the members of the upper lateral system are connected on the lower side of the connection plate in order not to interfere with the ties. Note that the use of the clip angles requires a smaller connection plate than would be necessary if these angles were not used, since in the latter case all the rivets must then be placed in one row in the horizontal leg of the angle.

The number of rivets required in the connection plate and the flange of the girder must be sufficient to take up the component of that member parallel to the girder. For the case in hand, the number (see Fig. 153) is:

$$\frac{x}{8} = \frac{6.2}{9.0},$$

from which,

$$x = 5.5 \text{ (say 6) rivets.}$$

Additional rivets should also be put in, in order to take up the component of the other lateral diagonal which meets at this point.

The member $U_1' U_1$ is to be designed for a maximum compressive stress of $20.6 + 0.8 \times 16.0 = -33.4$. A 6 by 4 by $\frac{7}{16}$-inch angle with an area of 4.18 square inches will be assumed. The least rectangular radius of gyration is 1.16. The unit allowable load is:

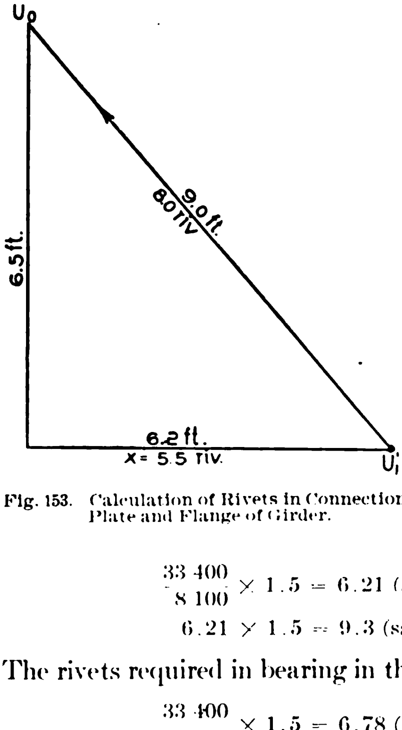


Fig. 153. Calculation of Rivets in Connection Plate and Flange of Girder.

$$P = 13\,000 - 60 \times \frac{88.6}{1.16} = 8\,420$$

pounds per square inch.

The required area is $\frac{33\,400}{8\,420} =$ 3.97 square inches. As this is very near the area assumed, and as trials with other angles do not give required areas which come any closer, this angle will be used.

The rivets required in single shear are:

$$\frac{33\,400}{8\,100} \times 1.5 = 6.21 \text{ (say 7) shop rivets, and}$$

$$6.21 \times 1.5 = 9.3 \text{ (say 10) field rivets.}$$

The rivets required in bearing in the $\frac{3}{8}$-inch connection plate are:

$$\frac{33\,400}{7\,380} \times 1.5 = 6.78 \text{ (say 7) shop rivets, and}$$

$$6.78 \times 1.5 = 10.17 \text{ (say 11) field rivets.}$$

The above computations show the joint to be weakest in bearing, and therefore 7 shop or 11 field rivets must be used. It is not necessary to investigate this member for tension, as the computations for the first diagonal indicate that the area will be sufficient, both legs being connected.

The member $U_1 U_2'$ must be designed for a maximum compressive stress of $16.0 + 0.8 \times 14.1 = -27.28$. A 6 by 4 by $\frac{3}{8}$-inch angle with an area of 3.61 square inches and a least rectangular radius of gyration of 1.17 will be assumed. The unit-stress P, as computed from the formula in the Specifications, is 8 460 pounds per square inch; and the required area is $\frac{27\,280}{8\,460} = 3.23$ square inches. This angle will be used, as the given and required areas are close together, and as the next smaller angle—a 6 by 3½ by ⅜-inch angle with an area of 3.42 square inches—gives a required area of 3.58 square inches, thus being too small.

The rivets required in single shear are:

$$\frac{27\,280}{8\,100} \times 1.5 = 5.04 \text{ (say 5) shop rivets, and}$$

$$5.04 \times 1.5 = 7.6 \text{ (say 8) field rivets.}$$

The rivets required in bearing in a ⅜-inch web are:

$$\frac{27\,280}{7\,380} \times 1.5 = 5.54 \text{ (say 6) shop rivets, and}$$

$$5.54 \times 1.5 = 8.3 \text{ (say 9) field rivets.}$$

In order to make the joints safe, 6 shop or 9 field rivets should be used.

The member $U_2'U_2$ must be designed for a maximum compressive stress of $9.6 + 0.8 \times 8.0 = -16.00$. A 3½ by 3 by ⅜-inch angle with an area of 2.30 square inches and a least rectangular radius of gyration of 0.90 will be assumed. The unit-stress P is 7 090 pounds per square inch, and the required area is $\frac{16\,000}{7\,090} = 2.26$ square inches. As the required and the actual areas are very close together, this angle will be used.

The rivets required in single shear are:

$$\frac{16\,000}{8\,100} \times 1.5 = 2.96 \text{ (say 3) shop rivets, and}$$

$$2.96 \times 1\tfrac{1}{2} = 4.44 \text{ (say 5) field rivets.}$$

By computation similar to the above, it is found that 4 shop or 5 field rivets are required in bearing. Since the bearing requires the most rivets to make the joint safe, 4 shop or 5 field rivets must be used.

If the Specifications would have allowed a 3½ by 3½ by 5/16-inch angle with an area of 2.09 square inches, this angle would have exactly fulfilled the requirements, the required area being 2.09 square inches.

The member U_2U_3' must be designed for a maximum compressive stress of $8.0 + 0.8 \times 4.1 = -11.28$. A 3 by 3 by ⅜-inch angle with an area of 2.11 square inches and a least radius of gyration of 0.91 will be assumed. In this case the unit-stress is 7 160, and the area required is 1.58 square inches. The required area is considerably less than the area of the angle assumed; but it must be used, since it is the smallest allowed by the Specifications, which require that the material shall not be less than ⅜-inch, and from Table XXI it is seen that 3 inches is the smallest size leg in which a ⅞-inch rivet can be used.

The stresses in all the members of the lower lateral system are less than the stresses in the member just designed, and therefore all members of the lower lateral system will be made of one 3 by 3 by ⅜-inch angle.

For the last member designed in the upper lateral system, and for all members in the lower lateral system, 3 shop or 5 field rivets will be required at the ends. These are more than sufficient to take up the stress, but it has been found that less than three rivets do not make a good joint.

The stress sheet, Plate II, shows the general arrangement of the lateral system, the number of rivets in the connections and also in the connection plates where they join the flanges.

The *intermediate cross-frames* do not lend themselves to a theoretical design, since the stresses which come upon them are not easily ascertained. It is good practice to require that all members be of the sizes as given below:

SPAN OF GIRDER (in Feet)	ANGLES (in Inches)	RIVETS	
		SHOP	FIELD
30 to 65	3½ x 3½ x ⅜	3	4
65 to 110	4 x 4 x ⅜	4	5

The angles in the intermediate cross-frames will therefore be 3½ by 3½ by ⅜-inch.

The *end cross-frames* (see Fig. 154) act in a manner somewhat similar to the portal bracing in a bridge, since they transfer all the wind which comes on the top chord and on the train to the abutment. This load, which acts at the level of the ties, is in this case (see Article 24 of the Specifications):

$$P = \frac{600 \times 61 \text{ ft. } 9 \text{ in.}}{2} = 18\,525 \text{ pounds.}$$

It is usually assumed that half of this is transferred to the point *a* by means of *a–b*, and from there down *a–b′* to the masonry. The other half goes directly down *b–a′* to the masonry. This causes stresses as shown in Fig. 154. Note that the stress in *a–b* will always be compression; but the stresses in the diagonal will be either tension or compression according to the direction the wind blows. The member *a–b* will be a 3½ by 3½ by ⅜-inch angle. To form the connections at its end, 3 shop or 5 field rivets will be used.

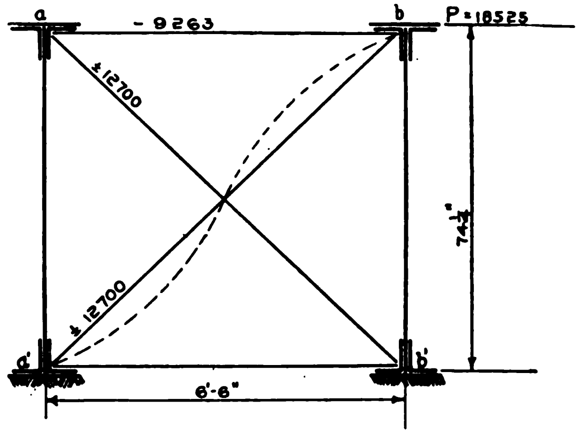


Fig. 154. Action of End Cross-Frames.

The maximum compressive stress for which the diagonals are to be designed is 12.70+0.8×12.70 = −22.86. Here the length is 108 inches if the angle tends to bend one way; but if it bends as shown by the broken lines in Fig. 154, the length will be one-half of this. For this reason, angles with unequal legs should be used, the longer leg extending outward. This allows the greatest rectangular radius of gyration to be used.

A 4 by 3 by $\frac{7}{16}$-inch angle with an area of 2.87 square inches and a radius of gyration of 1.25 will be assumed. The unit-load *P* is computed to be 8 750 pounds, and the required area is therefore 22 860 ÷ 8 750 = 2.61 square inches. This does not coincide very closely with the given area, but will be used since this angle comes nearer to fulfilling the condition than any of the other sizes rolled. The joint will require more rivets in bearing than in single shear. It is not necessary to perform the complete computations in order to determine this, since a comparison of the values of a rivet in single shear and in bearing shows that the value in bearing is less than that in single shear, and therefore the number of rivets required in bearing will be greater than that number required in single shear. The number of rivets required in bearing is:

$$\frac{22\,560}{4\,920 \times 1.5} = \text{say, 4 shop rivets, and}$$

$$4.00 \times 1.5 = 6 \text{ field rivets.}$$

76. **The Stiffeners.** According to Article 47 of the Specifications, these should be placed at certain intervals whenever the unit-shear is greater than

$$S = 10\,000 - 75 \times \frac{74}{\frac{3}{8}} = 10\,000 - 14\,800 = -4\,800 \text{ pounds}$$

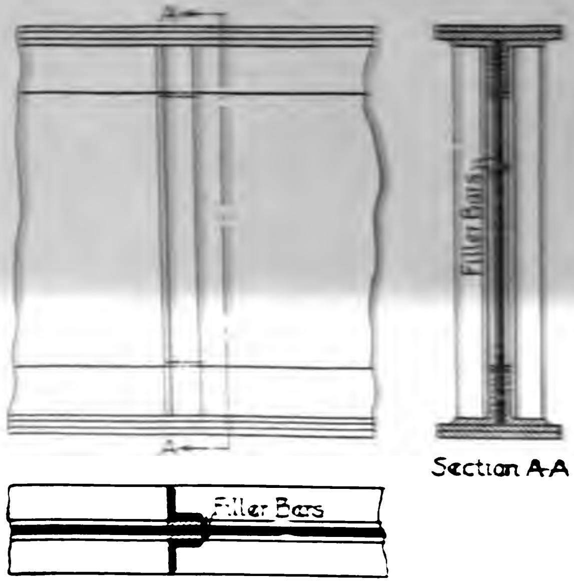


Fig. 155. Use of Straight Stiffeners with Filler Bars.

This negative sign signifies that whenever the unit shearing stress is greater than zero, the stiffeners must be placed throughout the entire length of the span at distances not to exceed 5 feet.

The intermediate stiffeners should have the outstanding leg long enough to give good support to the flange angle (see Fig. 155). The filler bars or fillers are put in so as to allow the stiffener angles to remain straight throughout their entire length; otherwise they will have to be bent as shown in Fig. 156. This bending is called *crimping*. Stiffeners must also conform to Article 48 of the Specifications. This would require a different sized stiffener at each point, and also a different number of rivets in each stiffener. This is not usually done in practice. In practice the stiffener for the first intermediate point is designed, and the remainder are made the same in size and have the same number of rivets. An exception to this is where a stiffener comes at a web splice. In this case the size is usually kept the same, but the number of rivets is changed somewhat to conform to the requirements of the splice design.

The second intermediate stiffener comes at the first tenth-point, and is 6.175 (say 6.2) feet from the end, since it is at the first panel point, or opposite the first panel point, of the lateral system. The first stiffener will be 3.1 feet from the end; and scaling off the value of the shear at this point (see Fig. 134), it is found to be 108 000 pounds. Here the length l to be used in the formula for the unit allowable compressive stress is $74\frac{1}{4} - 2 \times \frac{3}{4} = 72.75$ inches, the $\frac{3}{4}$ being the thickness of the flange angle. The section of the material which according to Article 48 of the Specifications is to be considered as a column, is shown in Fig. 157. The assumed column cannot bend about the axis $B-B$, but about the axis $A-A$, and therefore the radius of gyration about the axis $A-A$ must be computed. The moment of inertia of the fillers and the web plates about their own axes is considered as zero.

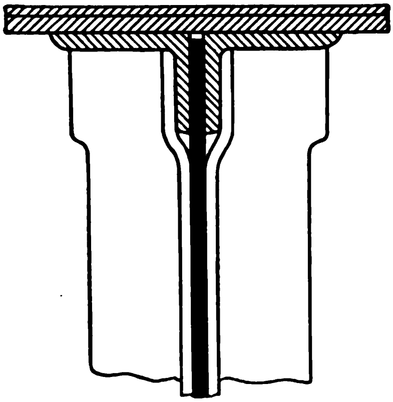


Fig. 156. Crimping of Stiffener Angles where No Filler Bars are Used.

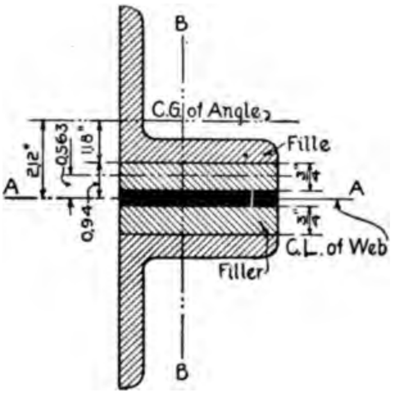


Fig. 157. Section of Intermediate Stiffener Construction.

A 4 by 4 by $\frac{1}{2}$-inch angle with an area of 3.75 square inches will be assumed to be sufficient to withstand the stress. The moment of inertia of this and the filler bars and the web portion is

$$I_{A-A} = 2\,(5.55 + 3.75 \times 2.12^2 + 3.00 \times 0.563^2) = 46.70.$$

The radius of gyration, then, is $\sqrt{\frac{46.70}{15.00}} = 1.764$, and the unit-stress computed with this value and a length of 72.25 inches is 8 140 pounds. The required area is now determined to be 108 000 ÷ 8 140 = 13.27 square inches. The value 15.00 used in the above computation for the radius of gyration is the value of the area of the angles, the filler bars, and the web portion. A 5 by 3½-inch angle with the 5-inch leg out would have given better support to the flange, but would not make so good a job, as it would have extended about ¼ inch beyond the curved part of the horizontal leg of the flange angle.

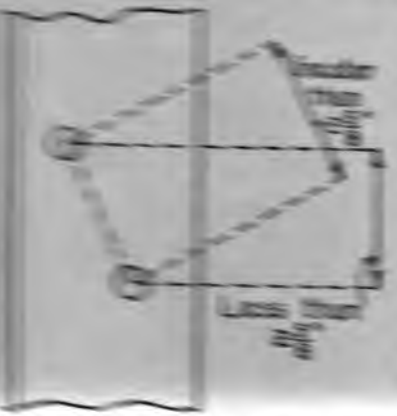


Fig. 158. Rivets Placed in Two Rows to Give Necessary Number and Spacing

The bearing determines the number of rivets in this case. The number is 108 000 ÷ 4 920 = 22 shop rivets in the web.

The angle must now be investigated in order to see if these 22 rivets can go in one row without being closer together than 2⅝ inches, which is three diameters of the ⅞-inch rivet. The total length in which these rivets must be placed is 72.25 inches, and therefore we have 72.25 ÷ 22 = 3.3 inches as a spacing. Since this is greater than 2⅝ inches, 22 rivets can be placed in one row. If the spacing as determined above had been less than 2⅝ inches, it would have been necessary to use two rows of rivets spaced as shown in Fig. 158; and then the distance center to center would be more than 2⅝ inches, although the spacing in a vertical line would be less than that.

The four angles at the ends of the girders are called the *end stiffeners*. These are placed in pairs on opposite sides of the web (see Plate II, Article 76).

The total end shear is 117 800 pounds, and this is assumed to be carried by the two pairs of end-stiffener angles, each carrying one-half. This amount would require lighter angles than the angles used for intermediate stiffeners. It is the customary practice to make them the same size and thickness as the intermediate stiffeners, additional strength being allowed in order to withstand the effects of the end cross-frame when in action.

The bearing determines the number of rivets required in each pair of stiffeners. The number required is $\frac{117\,800}{2 \times 4\,920} = 12$ shop rivets.

Some engineers arbitrarily choose the stiffeners regardless of the shear, enough rivets, however, being put in the end stiffeners to take up all the shear; and the spacing in the intermediate stiffeners is made the same. One noted engineering firm determines its stiffeners according to the following:

FLANGE ANGLE		STIFFENERS	
HORIZONTAL LEG	THICKNESS	END	INTERMEDIATE
4 in.	Any	3 x 3 x ½ in.	3 x 3 x ⅜ in.
5 in.	Any	4 x 4 x ½ in.	3½ x 3½ x ⅜ in.
6 in.	Over ¾ in.	4 x 4 x ½ in.	3½ x 3½ x ⅜ in.
6 in.	Less than ¾ in.	5 x 3½ x ½ in.	5 x 3½ x ⅜ in.
8 in.	Any	6 x 6 x ¾ in.	6 x 4 x ⅜ in.

No rational method has as yet been determined for ascertaining the stresses in the stiffeners of plate-girders. Results obtained by placing extensometers on the stiffeners of actual plate-girders appear to indicate that the stresses are very small, in fact in most cases not being greater than 1 500 or 2 000 pounds per square inch.

PROBLEMS FOR PRACTICE

1. Design, according to Cooper's Specifications, the end stiffeners if the shear is 150 000 pounds, the distance back to back of angles is 6 feet 6½ inches, the web ⅜ inch thick, and the flange angle 6 by 6 by ½-inch. Use fillers.

2. Design the intermediate stiffeners for the girder of Problem 1, above, where the shear is equal to 75 000 pounds. Use crimped stiffener angles. Note that in this case the angles lie close against the web, no filler bars being used in between.

76. The Web Splice. Web splices are required because of the fact that wide plates cannot be rolled sufficiently long. Web splices should be as few as possible, and good practice demands that they be placed at the same points as the stiffener angle.

The tables on page 30 of the Carnegie Handbook give the extreme length of plates which can be procured for any given width. The length of plates for widths which are not given in these tables, should be taken equal to the length of the next plate given whose width is less than that of the desired plate. From the first table it is seen that a 74 by $\frac{3}{8}$-inch plate can be rolled up to 400 inches, or 33 feet 4 inches, in length. Therefore, if the girder under consideration is spliced at the center, the web plates will be required to be $\frac{61 \text{ ft. } 9 \text{ in.}}{2} = 30$ feet $10\frac{1}{2}$ inches, which value does not exceed the 33 feet 4 inches as given above.

According to Articles 46 and 71 of the Specifications, a plate must be placed on each side of the web as shown in Fig. 159, and enough rivets placed in each side to take the total shear. The total thickness of both plates, and also their length, must be sufficient to stand the total shear, but must not be less than $\frac{3}{8}$ inch.

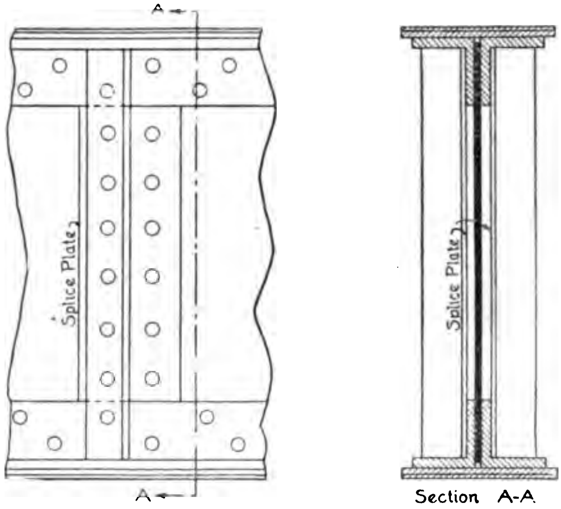


Fig. 159. Splice Plates Placed on Each Side of Web.

The total shear at the center of the girder under consideration (see Fig. 134, p. 150) is 28 600 pounds. The area required in each of the two splice plates is $\frac{28\,600}{2 \times 9\,000} = 1.59$ square inches; and as their length is 62.25 inches, the thickness must be $1.59 \div 62.25 = 0.0255$ inch, but they must be made $\frac{3}{8}$ inch thick according to the Specifications. The width should be somewhat greater than twice the width of the stiffener angle leg. This would make the width in this case about 10 inches.

The bearing governs the number of rivets required in this case, and they are $28\,600 \div 4\,920 = 5.81$, say 6, shop rivets. More rivets than this will be required by practical considerations, as indicated by

Article 54 of the Specifications or in order to make the spacing in the stiffener angle the same as that in the other stiffeners. This detail is to be left to the draftsman, the required number only being put on the stress sheet

In case ⅛ of the gross area of the web is considered as efficient flange area, then provision must be made in the splice for the bending moment which the web takes. A very economical and efficient splice is shown in Fig. 160. The horizontal plates take the stress due to the moment, and the vertical plates take the stress due to the shear.

The web equivalent is 3.47 square inches and the total moment is 1 615 000 pound-feet, which is composed of 275 000 pound-feet due to dead load and 1 340 000 pound-feet due to live load. Therefore that proportion of the 3.47 which is taken up by the dead load is:

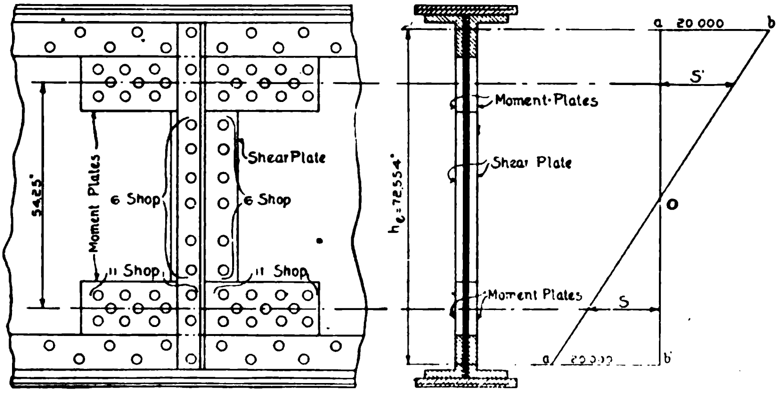


Fig. 160. Splice Consisting of Vertical and Horizontal Plates.

$$\frac{275\,000}{1\,615\,000} \times 3.47 = 0.59 \text{ square inch};$$

and that proportion taken up by the live load is:

$$\frac{1\,340\,000}{1\,615\,000} \times 3.47 = 2.88 \text{ square inches}.$$

The equivalent flange area is assumed to act at the center of gravity of the flange; and the bending moments equivalent to the above areas are, for dead load:

$$0.59 \times 20\,000 \times 72.554 = 856\,000 \text{ pound-inches};$$

and for live load:

$$2.88 \times 10\,000 \times 72.554 = 2\,090\,000 \text{ pound-inches}.$$

These bending moments must be taken up by the horizontal splice plates The stresses in these plates (see Fig. 160) are, for dead load:

$$\frac{856\,000}{54.25} = 15\,780 \text{ pounds;}$$

and for live load,

$$\frac{2\,090\,000}{54.25} = 38\,500 \text{ pounds.}$$

While the allowable unit-stresses are a maximum at the center of gravity of the flange and are those given by the Specifications, they decrease rapidly towards the center of the girder, being zero at the neutral axis of the entire section. The unit allowable stress at the center of the horizontal plates will not be so great as the maximum allowable, but will be proportional to the distance from center (see Fig. 160). The horizontal plates will be taken 8 inches in width. The unit-stresses are easily determined by means of the similar triangles *oab* and *oab'*. The dead-load stress is determined from the proportion:

$$\frac{S'}{20\,000} = \frac{\frac{54.25}{2}}{\frac{72.25}{2}},$$

and is 14 950 pounds. For live load, the unit-stress will be one-half of this amount, or 7 475 pounds.

The area required for this plate is, for dead load, $\frac{15\,780}{14\,950} = 1.05$ square inches, and for live load $\frac{38\,500}{7\,475} = 5.16$ square inches, making a total of 6.21 square inches for both plates. Assuming two rivet-holes out of the section, the net width is $8 - 2\left(\frac{7}{8} + \frac{1}{8}\right) = 6$ inches; and the required thickness for one plate is $\frac{6.21}{2 \times 6} \doteq 0.52$, say $\frac{9}{16}$ inch.

The joint will be weakest in bearing in the $\frac{3}{8}$-inch web. The number of rivets required is:

$$\frac{15\,670 + 38\,500}{4\,920} = 11 \text{ shop rivets.}$$

The design of the shear plate is as follows: The shear is 28 600 pounds, and the required area is $\frac{28\,600}{9\,000} = 3.18$ square inches. As the length of the plate is $46\frac{1}{4}$ inches, the required thickness is $\frac{3.18}{2 \times 46.25} =$

= 0.034 inch, but on account of the Specifications it cannot be less than $\frac{3}{8}$ inch thick. It will, however, be made $\frac{9}{16}$ inch thick, since it will then fill out even with the horizontal tension plates and no filler will be required. Bearing in the web plate decides the number of rivets, which is:

$$\frac{28\,600}{4\,920} = 6 \text{ shop rivets.}$$

The width of this shear plate should be, as before, 10 inches. The same conditions limiting the spacing of the rivets apply here as in the case where the splice was designed for shear only. The length of the horizontal plates should be sufficient to get in all the rivets, and this is a detail which is left to the judgment of the draftsman.

PROBLEMS FOR PRACTICE

1. A plate-girder is 87 ft. 9 in. center to center of end bearings. The dead-load moment is 9 125 000 pound-inches, and the live-load moment is 38 265 000 pound-inches, the total shear at the section being 202 700 pounds. The web is 90 by $\frac{7}{16}$-inch, and the flange angles are 6 by 6 by $\frac{3}{4}$-inch. Design the web splice when no part of the web is considered as taking bending moment.

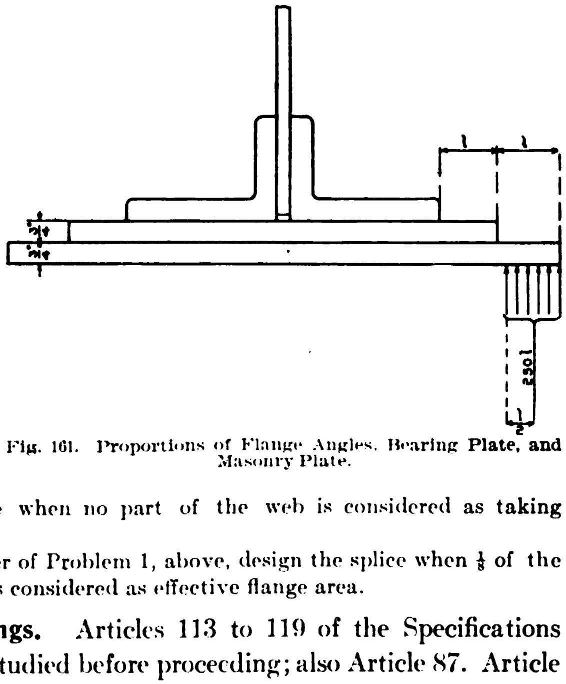


Fig. 161. Proportions of Flange Angles, Bearing Plate, and Masonry Plate.

2. For the girder of Problem 1, above, design the splice when $\frac{1}{8}$ of the gross area of the web is considered as effective flange area.

77. The Bearings. Articles 113 to 119 of the Specifications should be carefully studied before proceeding; also Article 87. Article 70 should be referred to, and the remarks there made about bearings should be read. In case the length of bearing is such as to allow a simple $\frac{3}{4}$-inch plate, care must be taken that the bearing plate does not extend past the flange angles more than 2 inches, or that the masonry plate does not extend past the bearing plate over 2 inches. Reference to "Steel Construction," Part II, p. 96, to Fig. 161, and to the discussion which follows, will explain the reason of this.

$$M = \frac{SI}{c} = 250 \times l \times \frac{l}{2};$$

$$I = \frac{bd^3}{12} = \frac{1 \times (\frac{3}{4})^3}{12};$$

$$c = \frac{d}{2} = \frac{\frac{3}{4}}{2} \qquad \therefore \frac{I}{c} = \frac{1 \times 9}{16 \times 12};$$

and as $s = 10\,000$,

$$250\,\frac{l^2}{2} = \frac{10\,000 \times 9}{16 \times 12},$$

from which,

$$l = 1.94, \text{ say 2 inches.}$$

In case it is desirable to have a simple masonry plate instead of a cast-steel pedestal, and to have the plate extend over the sides of the angles a distance greater than 2 inches, then some arrangement must be made for supporting the projecting portion. Fig. 162 shows one of the methods most commonly used. Notwithstanding the bracing of the gusset plates, the masonry plate is not adequately supported, the greater proportion of the stress coming upon the ends.

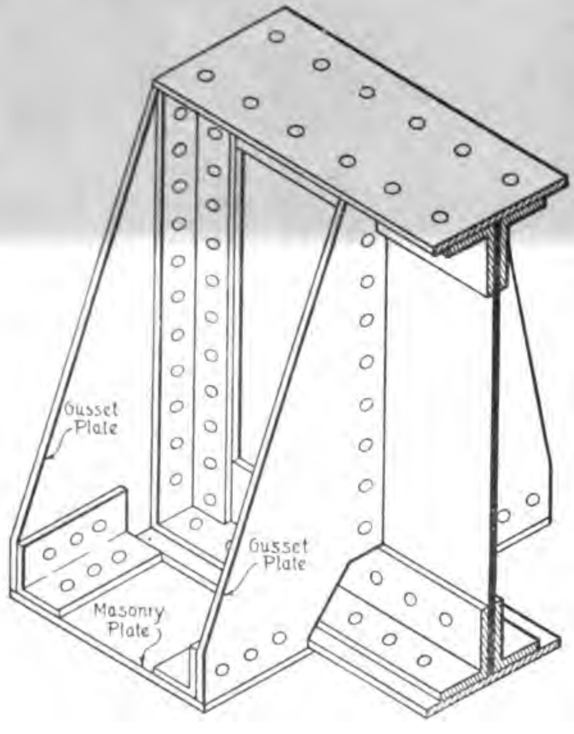


Fig. 162. Arrangement where Masonry Plate is Used instead of Cast-Steel Pedestal.

The disadvantage of having the masonry plate too long is plainly shown by Fig. 163. Here the girder is shown deflected under a live load, the rear end of the plate being tilted up and the greater part of the pressure coming upon the forward end. The use of this style of plate is not to be recommended for spans over 40 feet.

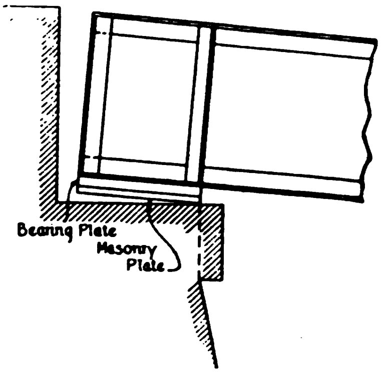


Fig. 163. Effect of Having Masonry Plate Too Long.

The design for the bearing of the girder under consideration will now be made. The total reaction of one girder must now be computed. This will be due to the weight of the steel in the girder, to the weight of the track, and to the reaction produced by the E 40 loading when wheel 2 is directly over the end support. This reaction is:

Weight of Steel, $\frac{(123.5 + 10 \times 61.75)\,61.75}{4} = 11\,430$ pounds

Weight of Track, $\frac{400}{2}\,(61.75 + 1.75)\,\frac{1}{2} = 6\,350$ "

Reaction Due to Engine Loading $= 99{\cdot}700$ "

Total $= 117\,480$ pounds.

The square inches of bearing surface required is $\frac{117\,480}{250} = 470$; and, as the length is 1 foot 9 inches, or 21 inches, the total width of the cast-steel pedestal will be $\frac{470}{21} = 22.4$, say 23 inches, or 1 foot 11 inches.

A bearing plate must be riveted to the lower flange where it rests upon the pedestal. The pedestal must be so constructed as to allow this bearing plate to set in it. Hand-holes should be provided in the casting in order to allow the bolts which connect the casting to the girder to be inserted. These bolts should be at least ¾ inch in diameter. Anchor bolts ⅞ inch thick and at least 8 inches long should be provided and fox-bolted to the masonry. The thickness of the metal in all parts of the casting should be at least 1¼ inches. The details of the pedestal are given in Fig. 164, the length of the bearing being made 12 inches so as to allow one rivet to be driven in the flange angle in the space between the end stiffeners.

Allowance should be made for a variation of 150 degrees in temperature. The coefficient of expansion for steel per unit of length is 0.0000065, and the amount of expansion for 150 degrees of temperature will be:

$$0.0000065 \times (61 \text{ ft. } 9 \text{ in.}) \times 150 = 0.06 \text{ foot.}$$

This is about $\frac{3}{4}$ inch, and therefore the holes in the flanges at one end of the girder should be made oblong and long enough to allow the girder to move $\frac{3}{4}$ inch, or $\frac{3}{8}$ inch either backward or forward from a central position. In determining the length of this slotted hole (see Fig. 165), it must be noted that the $\frac{7}{8}$-inch bolt takes up part of this hole, and therefore its length should be $\frac{7}{8} + \frac{3}{4}$ = say $1\frac{3}{4}$ inches. The width of the hole should be sufficient to allow for any over-run in the diameter of the bolt. It should be at least $1\frac{1}{4}$ inches wide.

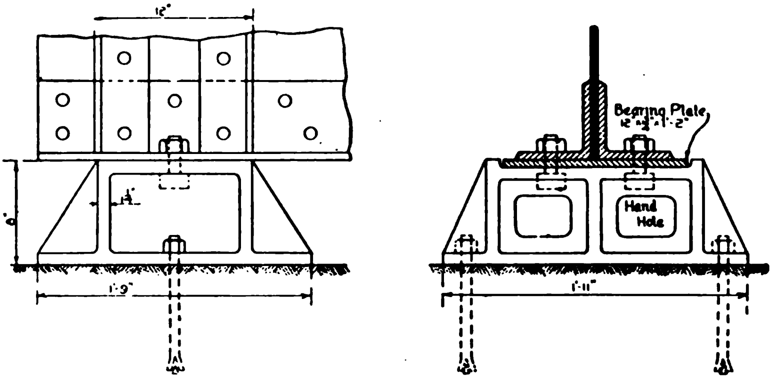


Fig. 164. Side and End Elevations Showing Construction of Pedestal and Connection to Bearing Plate.

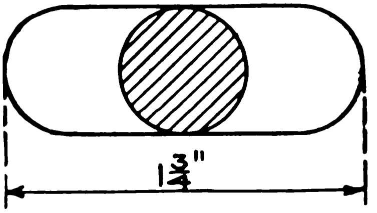


Fig. 165. Slotted Bolt-Hole in Flange at End of Girder to Allow for Contraction and Expansion Due to Temperature Changes.

PROBLEMS FOR PRACTICE

1. Determine the distance center to center of bearings, and the size of the masonry plate, for a plate-girder of 40-foot span under coping, the loading being E 40.

ANS. 41 ft. 4 in.; 350 square inches.

(NOTE—Interpolate values in Table I, Cooper, p. 30.)

2. If the girder span is 78 feet under coping, and the loading E 40, determine the maximum end reaction and the width of the masonry plate.

ANS. 147 130 pounds; 24½ inches.

78. The Stress Sheet. Plate II (p. 172) shows the stress sheet for

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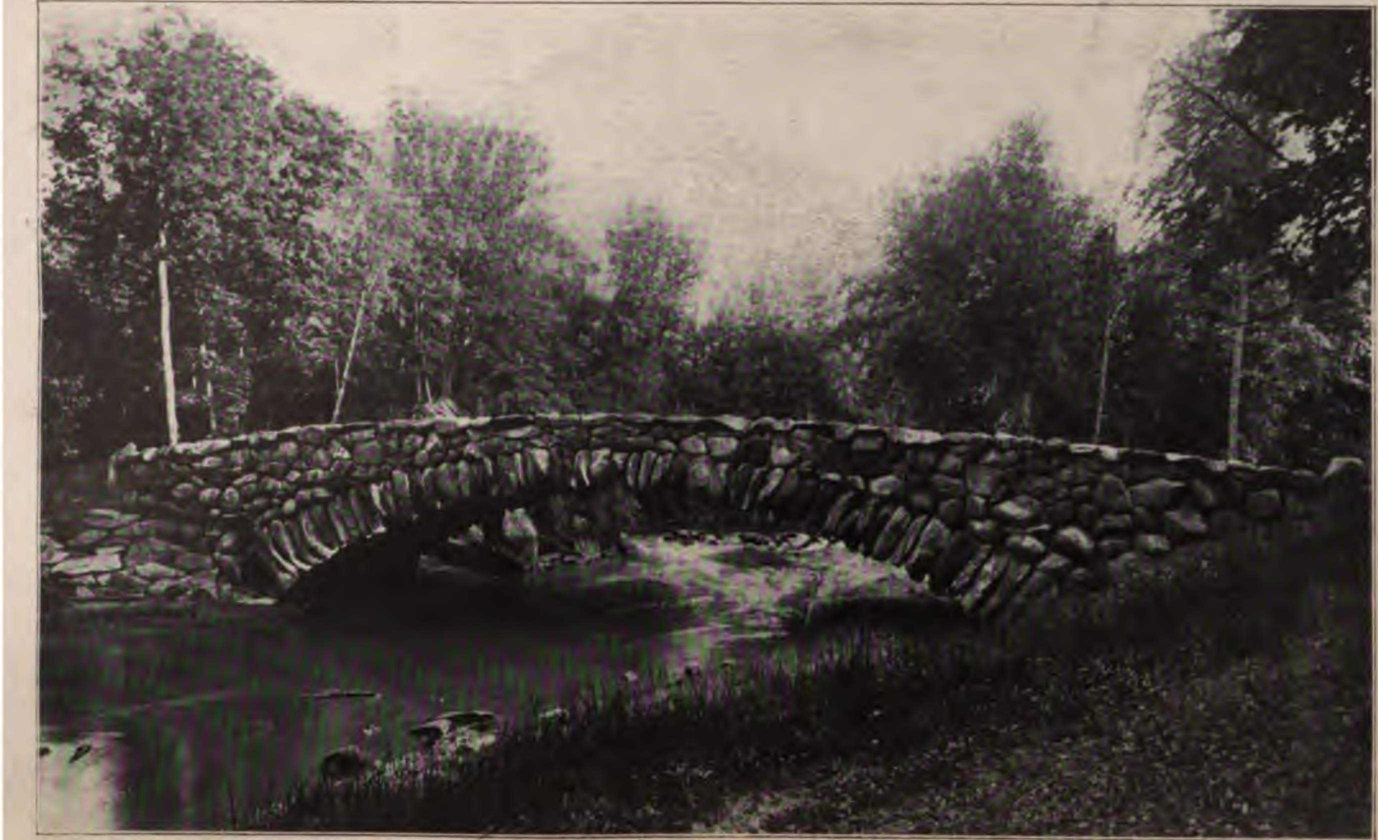

BOULDER BRIDGE IN ROCK CREEK PARK, WASHINGTON, D. C.

This structure is really a concrete-steel arch with a boulder facing. Each stone, dressed on the inner face, is attached to the adjacent steel or by a steel clamp cemented into a hole in the stone. The concrete of the soffit of the arch is artifically darkened to harmonize with the faces [illegible] a span of 80 ft., with a 15-ft. rise, and carries a 23-ft. roadway.

the girder which has just been designed. It represents the best modern practice in that it gives, in addition to the sizes of all the sections, the curves for the maximum shears and moments, the rivet-spacing curve, and the number of rivets required in the different parts of the structure. This general form has been adopted by one of the largest bridge corporations in this country, and is to be recommended since it gives the draftsman all necessary data and thus prevents the loss of time by an inexperienced man in recomputing certain results. The results just referred to are the shears, the moments, the rivet spacing, and the number of rivets required in the various parts. Formerly it was not customary to give this information on the stress sheet, and the draftsman was therefore required to do all this computation which had previously been worked out by the designer but had not been placed on the stress sheet in available form, and thus unnecessary loss of time resulted.

DESIGN OF A THROUGH PRATT RAILWAY-SPAN

79. The Masonry Plan. The same remarks which are made in Article 67 apply here. In this case the length of the masonry plate is usually determined by considerations relative to the number and length of the rollers in the bearing, and not by the bearing per square inch upon the masonry, the size of the plate as determined by the above considerations being usually much larger than if it had been determined by the unit bearing stress. A preliminary design of the masonry plate is usually made in a manner similar to that done in the case of the plate-girder; or the length of the masonry plate may be approximately determined from the following:

Span	Length of Masonry Plate	
	Fixed End	Roller End
100 feet	23 inches	23 inches
125 "	26 "	26 "
150 "	28 "	28 "
175 "	31 "	31 "
200 "	35 "	35 "

The above masonry plates are for single-track bridges, with or without end floor-beams, the length being the same in either case.

Determination of the Span. The determination of the span is made in exactly the same manner as described in Article 68. Care hould be taken, in case end floor-beams are not used, to allow for pedestal stones, which are square stones resting directly upon the seat, and upon the top of which rest the masonry plates of the Their height must, of course, be such as to keep the ngers level. In case these stones are used, their size must be determined; and if it is greater than that of the bearing or masonry plates, then their size determines the width of the bridge seat and the span center to center of bearings.

81. **The Ties.** In the design of the ties, as well as in all the design which follows, the Specifications of the American Railway

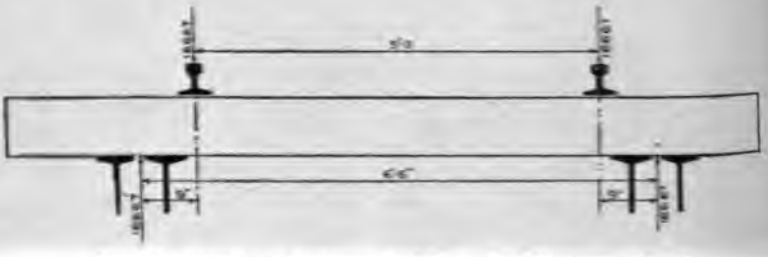

Fig. 166. Spacing of Stringers and Rails, and Position of Loads

Engineering & Maintenance of Way Association will be followed. Whenever reference is made to these specifications, the number of the article will be enclosed in parentheses, as "(5)," which signifies that Article 5 of the Specifications is to be referred to.

The stringers in the bridge in question will be taken 6 ft. 6 in. center to center. The maximum loading (7) is such as to bring 8 333 pounds on one tie, and to this must be added 100 per cent for impact, making a total of 16 667 pounds. In order to illustrate the method of assuming the distance, center to center of rails, as 5 feet, that distance will be used in this case. The maximum moment will then be 9 × 16 667 = say, 150 000 pound-inches. The size of the tie will be determined as in Article 71, the allowable unit-stress being 2 000 pounds per square inch (5). If a 7 by 9-inch tie is used, the unit-stress will be 1 590 pounds. If a 6 by 8-inch tie is used, the unit-stress will be 2 340 pounds. It is evident that a 7 by 9-inch tie must be used. See Fig. 166 for spacing of stringers and rails, and for position of the loads. Note that, although impact is taken into account in this case, the size of the tie is the same as that designed for the plate-girder, although the unit allowable stress also differs.

82. **The Stringers.** The width, center to center of trusses, will be assumed as 17 feet, since this is sufficient to clear the clearance diagram in cases of single-track bridges of spans less than 250 feet.

The span which is to be designed in the following articles is a through-Pratt with 7 panels of 21 feet each, making a total span, center to center of bearings, of 147 feet 0 inches. See Plate III (p. 251). Rivets $\frac{7}{8}$ inch in diameter will be used throughout, except in channel flanges.

The length of the stringers end to end will be 21 feet, and according to Cooper's Specifications, p. 32, the maximum moment for the live load will be 226 000 pound-feet per rail. The coefficient of impact (9) will be $\left(\frac{300}{21 + 300}\right) = 0.935$, and therefore the moment due to impact will be $0.935 \times 226\,000 \times 12 = 2\,535\,000$ pound-inches, making a total of 5 247 000 pound-inches due to live load.

The section modulus for any particular beam is equal to the bending moment divided by the unit-stress, and this is equal to the moment of inertia divided by one-half the depth of the beam. This latter quantity is constant for any given beam, and for I-beams may be found in column 11, Carnegie Handbook, p. 98.

On account of the cheapness of I-beams, they will be used for stringers in this bridge; and sufficiently heavy shelf angles will be used to take up any distorting influences due to the eccentric connections which are unavoidable in this case. In case an I-beam had not been decided upon, the stringers would have been small plate-girders with a span of 21 feet and depth according to formulæ given. They would have been computed in exactly the same manner as a plate-girder span of 21 feet center to center of bearings.

Since the dead-load moment cannot be determined until the size of the stringer is known, an approximate design must first be made by using the live-load bending moment alone; and then, with the size determined in this manner, the extra section modulus required for the dead-load moment due to the weight of the beam and the track must be computed. If this extra section modulus, added to the one previously determined, is greater than that given by the beam in tion, a larger size beam must be used and a recomputation made.

T e section modulus (17) required for live load only is $\frac{5\,247\,000}{16\,000}$

As this is too large for one beam, two beams will be used, g a required section modulus of 164 for one beam. Two pound I-beams will be used, giving a total section modulus 2 × 4 = 348.

Assuming the rails and ties to weigh 400 pounds per linear foot, the dead load per linear foot per stringer is $80 + \frac{400}{4} = 180$ pounds. The dead-load moment is therefore $\frac{180 \times 21 \times 21 \times 12}{8} = 119\,000$ pound-inches. This requires an additional section modulus of $\frac{119\,000}{16\,000} = 7.45$. This, added to the 164 as determined above, makes a total required section modulus of 171.25, which, being less than 174 (which is that for one I-beam), indicates that the above chosen beam is sufficient in strength, and it will therefore be used.

The number of rivets in the end connections will now be determined. The total end reaction for one I-beam is equal to the weight of one-half the beam, one-eighth the track in the panel, and one-half the maximum live-load reaction for one rail. These quantities are:

$\frac{1}{2}$ Live-Load Reaction $= \frac{51\,400}{2}$ $= 25\,700$ pounds.

Impact $= 25\,700 \times 0.935$ $= 24\,030$ "

Weight of Track $= \frac{400}{8} \times \frac{21}{2}$ $= 530$ "

Weight of Stringer $= \frac{21}{2} \times 80$ $= 840$ "

Total = 51 100 pounds.

The coefficient of impact is that for a loaded length of 21 feet.

From p. 177, Carnegie Handbook, sixth column, it is seen that the longest connection angle which can be used with a 24-inch I-beam is 20¾, say 20 inches. In this case the thickness of the connection angles must be $\frac{51\,100}{10\,000 \times 22} = 0.23$ inch; but according to (36), $\frac{3}{8}$ inch will be used. The angles chosen will be 6 by 3½ by $\frac{3}{8}$-inch. The 6-inch leg will be placed against the web of the floor-beam in order to allow for sufficient room for rivets to be driven.

The rivets will tend to shear off at places between the webs of the stringer and floor-beam and the connection angles. They will also tend to tear out of the web of the stringer and out of the web of the floor-beam. As the thickness of this latter is not known, the determination of the rivets for this condition will be made under the next article. The bearing value of a ⅞-inch rivet in a ½-inch plate (19) is $\frac{7}{8} \times \frac{1}{2} \times 24\,000 = 10\,500$ pounds, and therefore $\frac{51\,100}{10\,500} = 5$ shop rivets are required in bearing in the web of the stringer. The value of a ⅞-inch shop rivet in single shear (18) is $0.6013 \times 12\,000 = 7\,220$ pounds, and the number of rivets required to prevent shearing between the connection angles and the webs is $\frac{51\,100}{7\,220} = 7$ shop rivets. The value of a ⅞-inch field rivet in single shear (18) is $0.6013 \times 10\,000 = 6\,013$ pounds, and therefore $\frac{51\,100}{6\,013} = 9$ field rivets are required to connect the connection angle to the web of the floor-beam. As mentioned above, the number of rivets in bearing in the web of the floor-beam will be determined in the next article; and if the number required for bearing is greater than 9, then that number must be used instead of 9. Fig. 167 shows the connection of the stringer to the floor-beam web, and also the number of rivets as determined above, in their proper positions. The distance between the webs of the stringers must be such as to prevent their flanges from touching at the top.

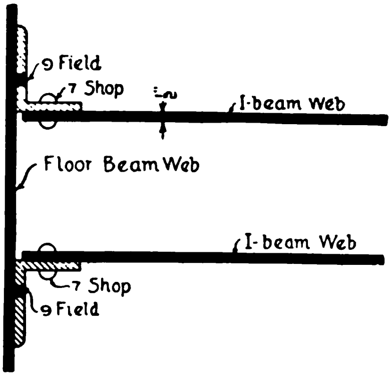

Fig. 167. Connection of Stringer to Floor-Beam Web; also Number of Rivets.

The stringers should be connected at the bottom by a system of lateral bracing of the Warren type. The size of these angles cannot be determined by theoretical considerations, but is usually chosen to be 3½ by 3½ by ⅜-in. See Plate II (p. 172) for the general arrangement of this bracing.

83. **The Floor-Beams.** All floor-beams should be of sufficient depth to allow the use of small-legged connection angles at the ends

where they join the end-posts. The thickness of the web should also be greater than that which is theoretically computed, in order that sufficient bearing may be given so that the rivets for the stringer connections will not require the stringers to be of too great a depth. The depth of the floor-beam will, of course, vary somewhat with the length of the panel and with the loading, but should not be less than 36 inches in any case. A considerable variation in the depth will not affect the weight of the floor-beam or the bridge to an appreciable extent. A good plan is not to exceed a depth of 5 feet, with panel lengths of 25 feet and E 50 loading. In this bridge the depth of all intermediate floor-beams will be taken as 48 inches. It is good practice not to consider ⅛ the web area when designing flanges of floor-beams and stringers, and the design here given does not consider the web as taking any bending moment.

The design of an intermediate floor-beam will now be made. The loads for which it is designed are the floor-beam reaction due to the live load (see Cooper, p. 32), the floor-beam reaction due to impact, the dead weight of the stringers and track, and the weight of the beam itself. The latter weight is distributed uniformly over the entire length of the beam, and the other loads act as concentrated loads spaced 6 feet 6 inches apart at equal distances from the center. The computation of the concentrated loads is as follows:

Live Load = ...	68 000 pounds
Impact $= 68\,000 \times \left(\frac{300}{37 + 300}\right)$	60 500 “
Dead Load of Stringer $= 2\left(\frac{21 \times 2 \times 80}{2}\right)$ ..	3 320 “
Dead Load of Track $= \frac{400}{2} \times 21$	4 200 “
	Total = 136 020 pounds.

The moment at points under the loads (see Fig. 168) is 136 020 × 5.25 × 12 = 8 575 000 pound-inches. This is due to the concentrated loads only. The weight of one floor-beam may be approximately determined by the same formula as used to determine the weight of plate-girder spans; only, in place of the length of the span, the length of the panel must be substituted. The total weight of the above floor-beam, then, is:

$$W = 0.45 \times (123.5 + 10 \times 21) \times 21 = 3\,160 \text{ pounds.}$$

The dead-load moment at the center due to this weight will be:

$$\frac{Wl}{8}=\frac{3\,160\times17\times12}{8}=80\,700 \text{ pound-inches,}$$

making a total moment at the center of the beam of 8 575 000 + 80 700

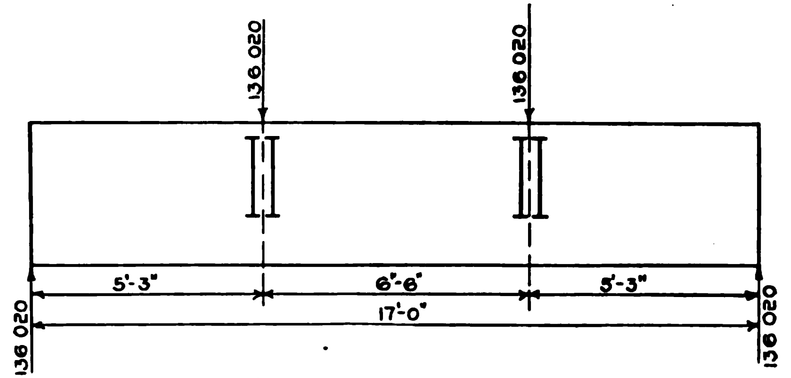

Fig. 168. Diagram Showing Loads on Floor-Beam.

= 8 655 700 pound-inches. Note that the dead-load moment at the center of the beam is added to the concentrated-load moment at the point where the concentrated load is applied. This will give the total moment at the center of the beam as shown by Fig. 169, since the concentrated-load moment is constant between the points of application. The end shear is readily computed to be 136 020 + 1 580 = 137 600 pounds. The curves of moments and shears are shown in Fig. 169.

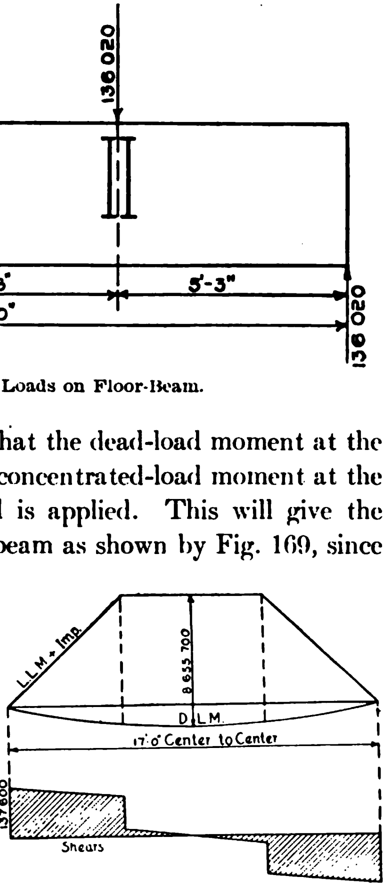

Fig. 169. Shear and Moment Diagram.

The total depth of the floor-beam, back to back of angles, is 48¾ inches; and the effective depth will, for approximate computation of the flange area, be taken as somewhat less, say 44½ inches, since the flange angles will probably be 6 by 6-inch and the center of gravity of most of these angles lies about 1¾ inches from the back. The approximate flange stress is $\frac{8\,655\,700}{44.5}=194\,500$ pounds, and the required net area (17) will be $\frac{194\,500}{16\,000}$

= 12.2 square inches. In assuming the size of the angle, it is to be remembered that when, as in this case, no cover-plates are used, no rivet-holes will be taken out of the top flange, and only one rivet-hole will be taken out of the vertical flange.

Two 6 by 6 by ⅝-inch angles give a gross area of 7.11 square inches each, and a net section of 7.11 − 0.625 = 6.485 square inches each, or 12.97 square inches net for both. As this is near the required area, these angles will be taken; and a recomputation will now be made with the actual effective depth, in order to see if sufficient variation in the areas occurs to require another angle to be taken. The actual effective depth is now 48¼ − 2 × 1.84 = 44.57 inches; and making computations with this, it will be found that a net area of 12.10 square inches is required. As this is practically the same as was determined at first, no change will be made in the size of the angle.

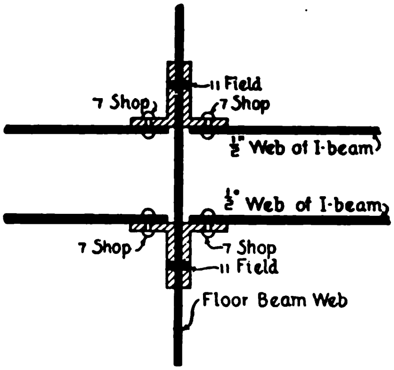

Fig. 170. Calculation of Number of Rivets through Connection Angles of Stringer and Floor-Beam Web.

The web is to be designed for a total shear of 137 600 pounds. The required area (18) is $\frac{137\,600}{10\,000} = 13.76$ square inches, and the required thickness is $\frac{13.76}{48} = 0.286$ inch; but on account of the Specifications (36), ⅜ inch must be used. The web will accordingly be 48 by ⅜-inch.

The determination of the number of rivets which go through the connection angle of the stringer and the web of the floor-beam can now be made. The value of a ⅞-inch field rivet in bearing in a ⅜-inch plate (19) is ⅞ × ⅜ × 20 000 = 6 560 pounds, and the total number required in one connection angle will be $\frac{136\,020}{2 \times 6\,560} = 11$ field rivets (see Fig. 170).

The pitch of the rivets in the flange can in this case be determined by the use of the formula:

$$s = \frac{vh_e}{V}.$$

Since the flange is of the same cross-section throughout, the value of the effective depth will not change, and it can therefore be used in the above equation instead of considering the value of the distance between rivet lines. The shear being practically constant from the connection of the stringers to the end of the floor-beams, the rivet spacing will be constant in this distance. It will be:

$$s = \frac{7\,880 \times 44.57}{137\,600} = 2.51 \text{ inches,}$$

the value of a $\frac{7}{8}$-inch shop rivet in bearing in the $\frac{3}{8}$-inch web being $\frac{7}{8} \times \frac{3}{8} \times 24\,000 = 7\,880$ pounds. This is seen to be less than $2\frac{5}{8}$ inches; but, as the angles have 6-inch legs, this spacing can be used in a horizontal direction; and the distance from center to center of rivets, which will be placed in rows on two gauge lines, will still be greater than $2\frac{5}{8}$ inches.

The shear between the stringer connections is practically zero, and therefore the spacing will be very large. Being over 6 inches, it will be subject to (37).

The connection angles at the ends of the floor-beams will be taken as 6 by $3\frac{1}{2}$ by $\frac{3}{8}$-inch, the 6-inch legs being against the web of the floor-beam. The other legs are chosen small in order that they may fit into the channels which will very likely be required for the posts; and according to the sixth column, p. 183, Carnegie Handbook, only $8\frac{1}{4}$ inches is available for this purpose. This $8\frac{1}{4}$ inches is taken from a 10-inch channel, since this is the smallest channel that can be used which will give room for connection and yet be in accordance with the Specifications. This is due to the fact that its web (36) is greater than $\frac{3}{8}$ inch. The rivets which connect the end angles to the floor-beam web are shop rivets, and those which connect the end angles to the posts are field rivets. Since the size of the post is not known, the thickness of its metal, of course, cannot be used, and therefore the number of rivets required in bearing in the post cannot be determined at this time.

The number of shop rivets required through the end angles and the floor-beam web is governed by the bearing of the rivets in the $\frac{3}{8}$-inch web of the floor-beam. The value of a $\frac{7}{8}$-inch shop rivet in bearing in a $\frac{3}{8}$-inch web (19), as has just been computed, is equal to 7 880 pounds, and the number of rivets required is $\frac{137\,600}{7\,880} = 18$.

The number of field rivets required in single shear to connect the end angles with the posts is $\frac{137\,600}{6\,013} = 23$. An even number of rivets will, of course, have to be used, one-half going into each of the $3\frac{1}{2}$-inch legs. See Fig 171 for the position and the number of rivets. It must be remembered that more than these numbers may be used by the draftsman on account of rivet spacing which may be required by conditions other than those of design.

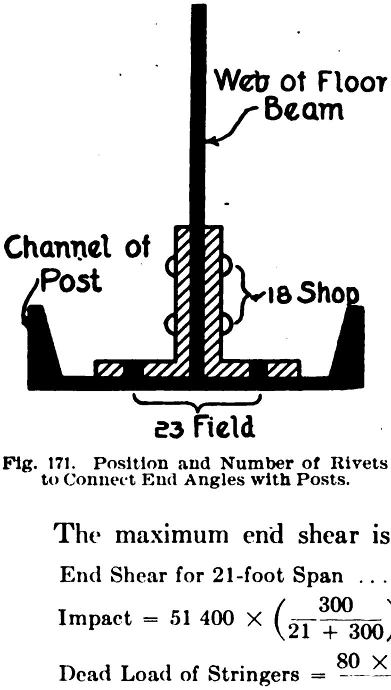

Fig. 171. Position and Number of Rivets to Connect End Angles with Posts.

The design of the end floor-beam is somewhat different from that of the intermediate floor-beams in that the load which comes upon it is considerably lighter, since this floor-beam takes the dead load of only one-half the panel and the live load due to the *maximum end reaction* for a stringer length instead of the floor-beam reaction for the stringer length (see Cooper, p. 32).

The maximum end shear is computed as follows:

End Shear for 21-foot Span	51 400	pounds.
Impact = $51\,400 \times \left(\frac{300}{21 + 300}\right)$	48 000	"
Dead Load of Stringers = $\frac{80 \times 2 \times 21}{2}$	1 680	"
Dead Load of Track = $\frac{400 \times 21}{2 \times 2}$	2 100	"
Total	103 180	pounds.

The maximum moment due to the above load is $103\,180 \times 5.25 \times 12 = 6\,500\,000$ pound-inches. The weight of the beam may be assumed as 3 160 pounds. This is the same as was computed for the intermediate floor-beam, but will be used for this beam, since the size of the web will be the same as in the others; and, although the flange area will be less, the end connections will be somewhat heavier owing to the connection of the beam to the end-post and the roller bearing, and this additional weight will cause the total weight of the end floor-beam to be about the same as that of the intermediate ones. The total moment at the center will then be 6 500 000 + 80 700 = 6 580 700 pound-inches.

The depth of the end floor-beam will be somewhat greater than the depth of the intermediate floor-beams. This is due to the fact that it extends downward a greater distance, resting upon the bearing plate, which comes directly upon the top of the rollers. The exact depth cannot, of course, be determined until after the roller bearings are designed; but it may be safely assumed as four or five inches deeper than the intermediate floor-beams, and in case this is not enough, the draftsman can easily fill in the remaining distance with filler plates, as this distance will not be very great. In case this depth is too great, the flange angles may be bent upward at the end, or a re-design may be made.

The depth will be assumed as 52 inches in this case. The effective depth will be assumed as 48 inches, and this gives an approximate flange stress of

$$\frac{6\ 580\ 700}{48} = 137\ 000 \text{ pounds,}$$

and an approximate net flange area required of

$$\frac{137\ 000}{16\ 000} = 8.57 \text{ square inches.}$$

A 6 by 6 by $\frac{7}{16}$-inch angle gives a gross area of 5.06 square inches, and a net area of $5.06 - (\frac{7}{8} + \frac{1}{8})\ \frac{7}{16} = 4.62$ square inches. A recomputation with the true effective depth requires 8.42 square inches net. Two of these angles give 9.24 square inches; and as this coincides very closely with the required area, it will be used. The size of the web plate is 52 by $\frac{3}{8}$-inch.

The pitch or spacing of rivets in the flanges is:

$$\frac{7\ 880 \times 48}{104\ 760} = 3.62 \text{ inches.}$$

The maximum end shear as above computed is taken by two stringers; and therefore the number of rivets required in bearing to form the connection between the stringers, connection angles, and the floor-beam web is, for each angle:

$$\frac{104\ 760}{2 \times 6\ 560} = 8 \text{ field rivets.}$$

[illegible] the above equation is the value of a $\frac{7}{8}$-inch field [illegible] the $\frac{3}{8}$-inch web.

[illegible] of rivets required in the end angles on the floor- [illegible]

$$\frac{104,780}{7,880} = 14 \text{ shop rivets.}$$

[illegible] through the web of the floor-beam. The connection [illegible] floor-beam to the end-post is made by means of field rivets and a large gusset plate. This gusset plate is usually $\frac{3}{8}$ inch in thickness. The number of rivets through the end connection angles and this gusset plate is governed [illegible] single shear, since the rivets will shear off between the angles and the gusset plate before they will tear out of the gusset plate, as the value of a rivet is greater in bearing than in shear. The number required is:

$$\frac{104,780}{6,013} = 18 \text{ field rivets.}$$

The general arrangement of the intermediate floor-beams is shown in Fig. 172. The ends of the lower flange are bent up as shown, in order to allow the I-bar heads or any other section of the lower chord to have clearance. This makes it necessary for the floor-beam web to be spliced at the ends, as shown. The distance which [illegible] floor-beam proper depends upon [illegible] is bent up. In any case the length [illegible] should be at least equal to the depth [illegible] plates, one on either side of the web, [illegible] similar to that of a splice as designed in [illegible] was considered. Here shear only is [illegible] rivets which must be on each side of [illegible]

[illegible] shop rivets.

[illegible] equation is the value of a $\frac{7}{8}$-inch [illegible] plate [illegible] Inspection of Plate II (p. [illegible] clearer. Plate II also shows the shape of [illegible] end floor-beams.

The small shelf angle shown in Fig. 172 should have sufficient rivets to prevent any twist of the stringers due to their being connected on one side of their web only. This number is a matter of judgment. Experience seems to indicate that enough rivets to take up one-third of the total reaction of the stringers will be sufficient. This will require shop rivets, and the number will be:

$$\frac{103\,180}{3 \times 7\,220} = 5 \text{ shop rivets in single shear.}$$

84. **The Tension Members.** Tension members usually consist of long, thin, flat plates with circular heads forged upon their ends.

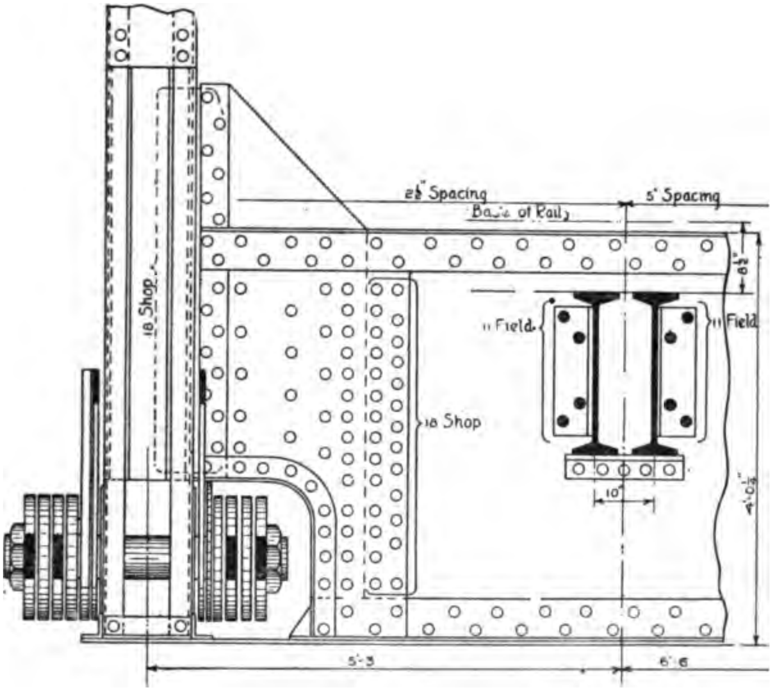

Fig. 172. General Arrangement of Rivets, Splices, Connections, etc., for Intermediate Floor-Beams.

These circular heads have holes punched through their centers and then very carefully bored. Through these holes are run cylindrical bars of steel called *pins*. These pins connect them with other members of the truss. See Carnegie Handbook, p. 212, for table of I-bars. The I-bars given are standard I-bars; and while departures from these widths and *minimum* thicknesses may be made, it may be done only at great cost to the purchaser. Note that there are no standard 9-inch I-bars. The thicknesses given are the *minimum* thicknesses [illegible] enable is [illegible] if a given [illegible] the actual [illegible] the material [illegible]

[illegible] the bar will [illegible]

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TABLE XXIV

Tension Members

MEMBER	STRESSES (in pounds)					REQUIRED AREA*	NO. OF BARS	AREA*; ONE BAR	$d = \sqrt{6A}$	SECTION USED*	AREA* OF SECTION USED
	DEAD-LOAD	LIVE-LOAD	IMPACT	TOTAL	UNIT						
L_0L_2	41 000	114 700	78 500	234 200	16 000	16.64	..	....	....	......	....
L_1U_1	13 000	68 000	60 500	141 600	16 000	8.85	..	....	....	......	....
L_2U_1	47 900	143 900	103 300	295 100	16 000	18.44	2	9.22	7.44	7 x $1\frac{3}{8}$	9.64
L_2U_3	......	28 000	23 100	25 900	16 000	1.62	1	1.62	....	$1\frac{3}{8}$ x $1\frac{3}{8}$	1.93
L_3U_2	24 000	95 200	72 600	191 800	16 000	11.98	2	5.99	6.00	6 x 1	6.00
L_2L_3	68 400	183 700	126 100	378 200	16 000	23.60	4	5.90	5.95	6 x 1	6.00
L_3L_4	82 000	218 400	149 100	449 500	16 000	28.20	4	7.05	6.50	6 x $1\frac{1}{4}$	7.50
L_3U_4	......	54 200	44 100	98 300	16 000	6.14	2	3.07	4.30	4 x $\frac{7}{8}$	3.50

* NOTE—All areas in square inches, and all dimensions in inches.

Observe that the above table is not completely filled out with respect to the first two members given. This is on account of the requirements of the Specifications (80).

therefore an excessively large pin would be required, and accordingly a very large head on the I-bar in proportion to its width—all of which are very undesirable and costly. In general the number of I-bars should be as small as possible, and they should be so chosen that the widths of the chord members *increase* from the ends toward the center of the truss, and the widths of the diagonals *decrease* from the ends toward the center of the truss.

The area of one bar is obtained by dividing the total stress by the number of bars and also by the allowable unit-stress. Thus, for the member L_2U_1, for example, the required area of one bar is:

$$\frac{295\,100}{2 \times 16\,000} = 9.22$$

square inches.

The approximate depth of this bar is determined by taking the square root of 6 times the area as above determined. It is:

$$d = \sqrt{6 \times 9.22} = 7.44 \text{ inches.}$$

As this is nearer 7 than 8 inches, a 7-inch bar will be chosen; and looking in the first column, Carnegie Handbook, p. 248, for an area which will be equal to or in excess of 9.22, it is found that a 1⅜-inch bar satisfies this condition, and therefore the section of this member consists of two bars 7 by 1⅜ inches.

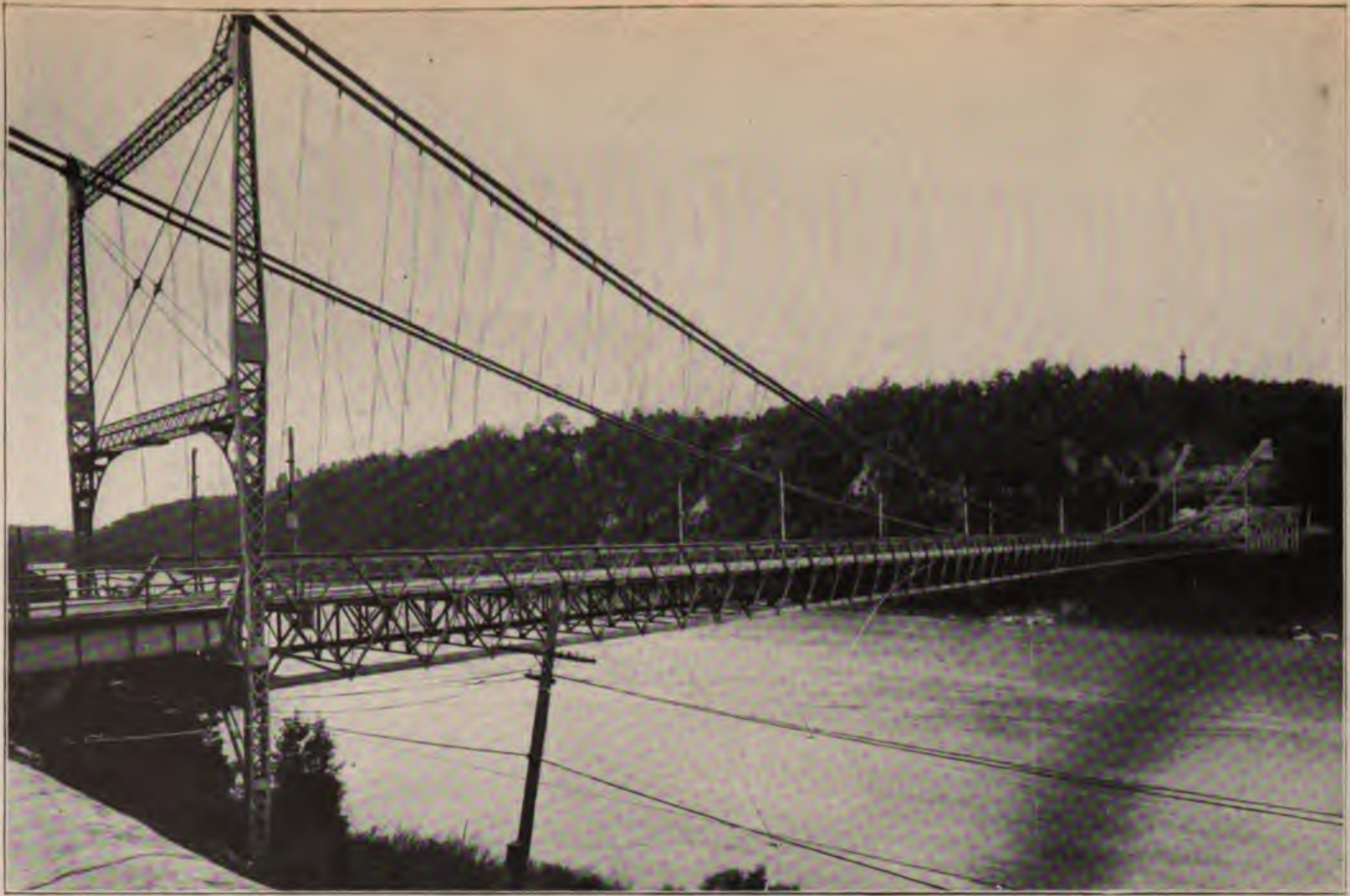
According to (80), the first two sections for the lower chord are to be made of built-up members. This requires that instead of I-bars they are to be made of angles and plates, or, in case the stress is light, of channels. The depth of the section is limited by the size of the greatest I-bar head. As the diameter of the I-bar head depends upon the size of the pin, it cannot of course be determined accurately before the pin is designed. It is customary to assume the largest head, and to design the section so as to clear this. The size of the largest head for bars of given width is given in the Carnegie Handbook, p. 212.

The design of the member L_0L_2 will depend upon the size of the largest head of the 7-inch I-bar of the member U_1L_2. This is 17¼ inches; and in order that the head may have some clearance, it will be necessary to add ½ inch to the top and the bottom, making a total of 18¼ inches. Since the flange angle, as in the case of plate-girders, will extend over the plate about ¼ inch, the plate itself may be 18 inches wide and still give sufficient clearance.

The total stress is 234 200 pounds, and the allowable unit-stress (15) is 16 000 pounds per square inch. The required net area, then, is:

$$\frac{234\,200}{16\,000} = 14.64 \text{ square inches.}$$

According to the Specifications, the thickness of the plate cannot be less than ⅜ inch. The gross area of two 18 by ⅜-inch plates is 13.5 square inches, and the gross area of four 3½ by 3½ by ⅜-inch angles, which are assumed to be sufficient, is 9.92 square inches, thus making a total gross area of 23.42 square inches. If 5 rivet-holes are assumed to be taken out of each web, and one rivet-hole taken out of each angle, this will require a certain number of square inches to be



SUSPENSION BRIDGE OVER THE NIAGARA RIVER AT LEWISTON, N. Y.

Built in 1899. Span length of stiffening truss, 800 ft.; span between main towers, 1,040 ft.; distance between anchorages, 1,200 ft.

Courtesy of American Bridge Company.

deducted from the section, and this is computed as follows:

Out of webs, $2 \times 5\left(\frac{7}{8} + \frac{1}{8}\right) \times \frac{3}{8} = 3.75$ sq. in.
Out of angles, $4\left(\frac{7}{8} + \frac{1}{8}\right) \times \frac{3}{8}$ . . $= 1.50$ " "
Total $= 5.25$ sq. in.

The net area of the section is now determined to be $23.42 - 5.25 = 18.17$ square inches. This is somewhat greater than the required net area, but must be used, for according to (39), these are the smallest and thinnest angles that may be used.

Figs. 173 and 174 show the cross-section and the general detail at L_2. The width of the member cannot be determined until after the section of the end-post is computed, since it must fit inside of the end-post, the horizontal legs of the angles being cut off to allow this. The end-post, Article 87, is $14\frac{1}{2}$ inches inside. If it is assumed that all the pin-plates on the end-post are placed on the outside, and all those at L_0 on L_0L_2 are on the inside, then the width of L_0L_2, back to back of plates, must be $14 - \left(2 \times \frac{1}{2} + 2 \times \frac{3}{8}\right) = 12\frac{1}{4}$ inches or less, $\frac{1}{8}$-inch clearance being allowed between the sides of the angles and the web plates of the end-post (see Fig. 173).

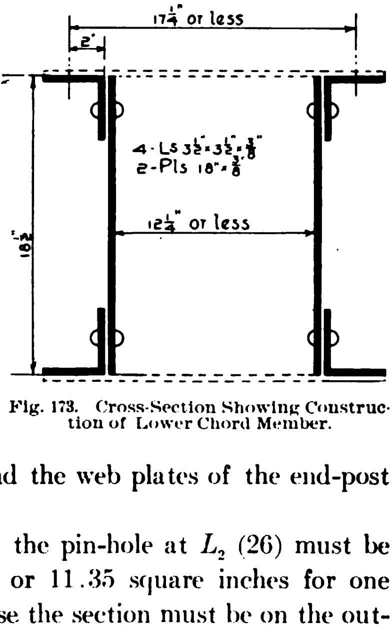


Fig. 173. Cross-Section Showing Construction of Lower Chord Member.

The total net section through the pin-hole at L_2 (26) must be $1\frac{1}{4} \times 18.17 = 22.7$ square inches, or 11.35 square inches for one side. The plate which is to increase the section must be on the outside, since the intermediate post U_2L_2 and the two I-bars of member U_1L_2 must go inside. The gross width of this plate is $11\frac{1}{4}$ inches (see Fig. 174), and the net width is $2w = 11\frac{1}{4} - 5 = 6\frac{1}{4}$ inches. The net area through the pin is:

Two $3\frac{1}{2}$ by $3\frac{1}{2}$ by $\frac{3}{8}$-in. angles $= 4.96$ square inches.
One 18 by $\frac{1}{2}$-in. plate $= 9 - 5 \times \frac{1}{2} = 6.50$ " "
Total $= 11.46$ square inches

Since this is greater than the 11.35 required, no plate will be necessary to fulfil (26) in this respect.

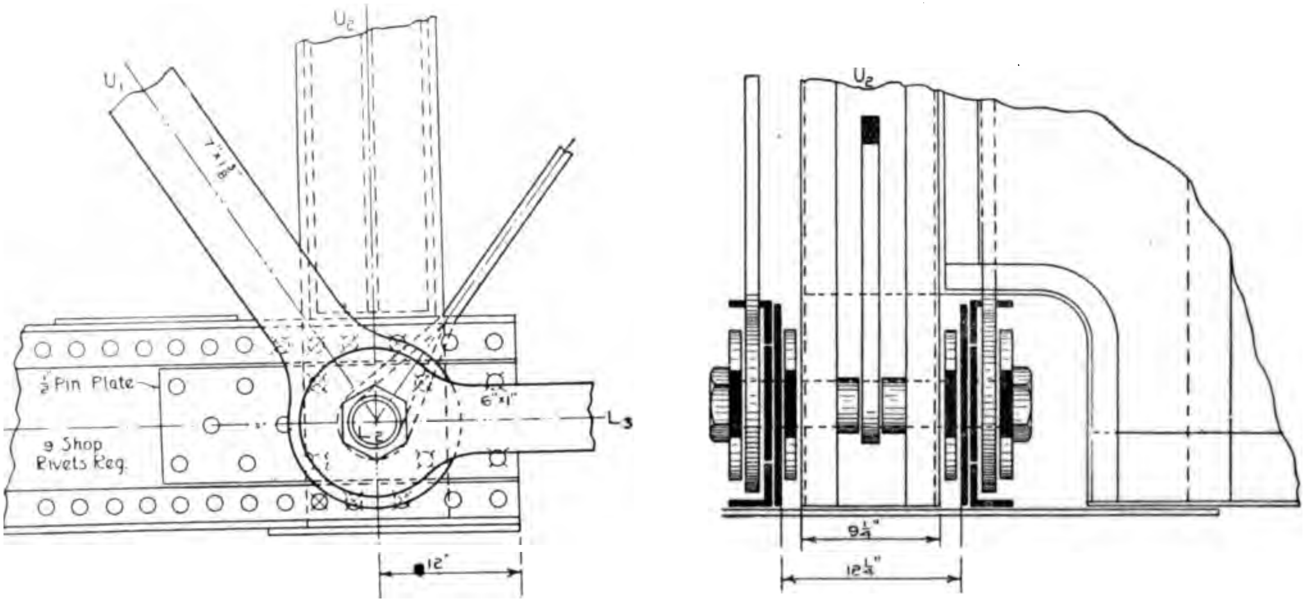


Fig. 174. Side Elevation and Transverse Section Showing General Details of Construction at Panel Point in Lower Chord.

Sufficient bearing area must be provided at this point. The total stress is 234 200 pounds, the total bearing area required is $\frac{234\,200}{24\,000} = 9.76$ square inches, and the total thickness for one side is $\frac{9.76}{2 \times 5} = 0.976$ inches. Since the thickness of the web is ½ inch, the pin-plates must be $0.976 - 0.50 = 0.476$ inch (say ½ inch) thick. A ½-inch pin-plate must be used, and as the total thickness of the bearing area is now 1.00 inch, this pin-plate will take $\frac{0.50}{1.00} \times \frac{234\,200}{2}$ $= 58\,550$ pounds. The joint is weakest in shear, and will therefore require $\frac{58\,550}{7\,220} = 8 +$ (say 9) shop rivets.

In case it is necessary to put the member U_1L_2 on the outer side of L_0L_2, then the outer legs of the upper angles must be cut off to allow U_1L_2 to pass. This will decrease the section by an amount $(3\frac{1}{2} - \frac{3}{8}) \times \frac{3}{8} = 1.20$ square inches. Considering the pin-plate, which is $(18\frac{1}{2} - 2 \times 3\frac{1}{2}) - \frac{1}{4} = 11\frac{1}{4}$ inches, the ¼ inch being allowed for clearance between the edges of its flange angles, the total net section through the pin-hole on one side will be:

One Angle 3½ by 3½ by ⅜-in.	= 2.48	square inches
One Cut Angle 3½ by 3½ by ⅜-in.	= 1.31	" "
One Web (18 − 5) ½ sq. in.	= 6.50	" "
One Pin-Plate (11¼ − 5) ⅜ sq. in.	= 2.34	" "
Total	= 12.63	square inches.

This is greater than 11.35 as required, and is therefore safe.

The distance from the center of the pin to the end must now be determined (26). The total net section of the body of the member is 18.17 square inches, or 9.09 square inches for one side, and the thickness of the web and the pin-plate is 1 inch. The distance from the pin to the end of the member is then $\frac{9.09}{1} = 9\frac{1}{8}$ inches, and the distance to the center of the pin is $9\frac{1}{8} + \frac{5}{2} = 11\frac{5}{8}$, say 12 inches (see Fig. 174). Rivets should be countersunk where necessary to prevent interference with I-bars. For signs, see "Steel Construction," Part III, p. 192, and Carnegie Handbook, p. 191.

At point L_0 of this member, the pin is 6¼ inches in diameter, and, as previously mentioned, the legs of the angles are cut (see Fig. 175). The total bearing area required for one side is $\frac{9.76}{2} = 4.88$, and the required thickness is $\frac{4.88}{6.25} = 0.781$ inch. Subtracting the thickness of the ½-inch web from this gives 0.281 inch. A pin-plate ⅜ inch thick must be used.

The net area through the pin (26) must be 11.35 square inches.

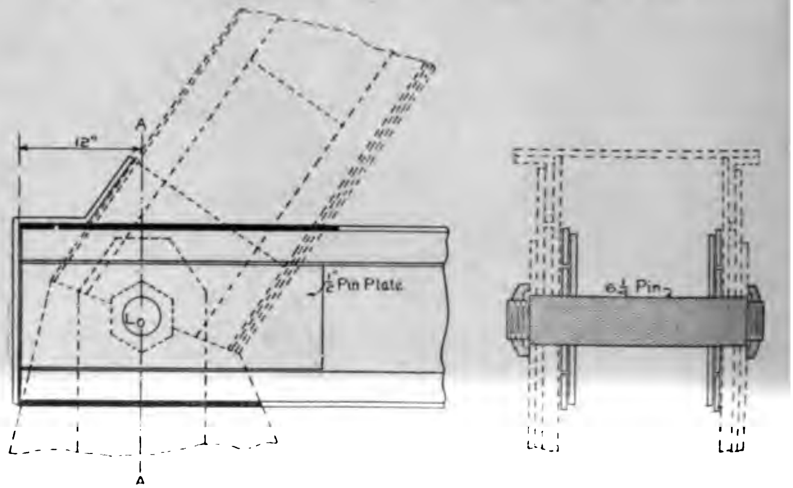


Fig. 175. Elevation and Section Showing Pin Connection at End of Truss.

This net area, remembering that the angle legs are cut and therefore their area is that of a bar 3½ by ⅜-inch, computed for one side, is as follows:

Two Angles, legs cut, 3½ by 3½ by ⅜-in.	=	2.62	square inches
One Web 18 × ½ − 6¼ × ½ sq. in.	=	5.88	" "
One Pin-Plate 11¼ × ⅜ − 6¼ × ⅜ sq. in.	=	1.87	" "
Total		10.37	square inches.

This shows the section to be deficient, and the thickness of the pin-plate must be made ½ inch. This gives a net area through the pin of 11.62 square inches.

The distance between rivet lines (see Fig. 173) is 17¼ inches, and (44) the tie-plates must be 17¼ (say 18) inches long, and their thickness $\frac{17.25}{50} = 0.346$ inch (say ⅜ inch).

The lattice bars (45) must be 2½ inches wide, and (47) must be double. From (45) and Table XXV, page 219, the thickness must be $\frac{7}{16}$ inch, the distance c being 17.25 × secant 45° = 2 ft. 0$\frac{7}{16}$ in.

The design of the hip vertical U_1L_1 is also made in accordance with (80) of the Specifications. It will be assumed that the section consists of one 8 by ⅜-inch plate, and four 3½ by 3½ by ⅜-inch angles, since this is the lightest section that may be used according to the Specifications, the 8-inch plate being chosen as it gives some clearance between the inner edges of the legs of the angles.

The total stress in the member is 141 600 pounds, and the unit-stress is 16 000 pounds per square inch, thus requiring a net area of 8.85 square inches. The plate gives a net area of 2.20 square inches, and the four angles give a net area of 6.88 square inches, making a total of 9.08 square inches, one rivet-hole being taken out of each angle, and two out of the web, at any particular section. The net area is somewhat greater than that required, but must be used, as this is the minimum section allowed by the Specifications. Fig. 176 shows a cross-section of this member as above determined.

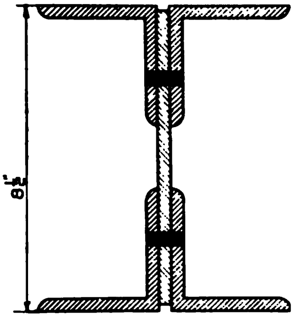


Fig. 176. Cross-Section of Hip Vertical.

This member will be connected to the upper chord and end-post by means of a pin which is 6¼ inches in diameter, the diameter of the pin being determined later. The total stress is 141 600 pounds, and this will be taken by two plates, one on either side of the member. The net section of the member is 9.08 square inches, and the section through the pin (26) must be 25 per cent in excess of this, making a total of 11.35 square inches, or 5.68 square inches for each plate. The total width of these plates will be taken as 12 inches, and this (see Fig. 177) will make the required thickness:

$$\frac{5.68}{2c} = \frac{5.68}{12.00 - 6.25} = 0.99 \text{ inch.}$$

Fig. 177 shows the details of these pin-plates. Since the above thickness is too great to be punched in one single piece, the above thickness will be made up of two plates, each ½ inch thick. The area at section A–A must be equal to that of the body of the bar. It is 12.00 × ½ = 6.00 square inches for one side, or 12.00 square inches for both sides. As this is greater than the 8.85 square inches as above computed, the area at *A–A* is sufficient, as is also the width of the plate, which was assumed as 12 inches.

One of the plates will be riveted directly to the member, and the other will be riveted to it as a pin-plate. The section back of the pin (26) must be equal to the net section in the body of the member. The net section is 4.54 square inches for one side, and the total thickness of the pin-plates is 1.125 inches, making the distance from the end of the member to the pin $\frac{4.54}{1.125} = 4\frac{1}{8}$ inches, and the distance to the center of the pin $4\frac{1}{8} + \frac{6\frac{1}{4}}{2} = 7\frac{1}{4}$ inches.

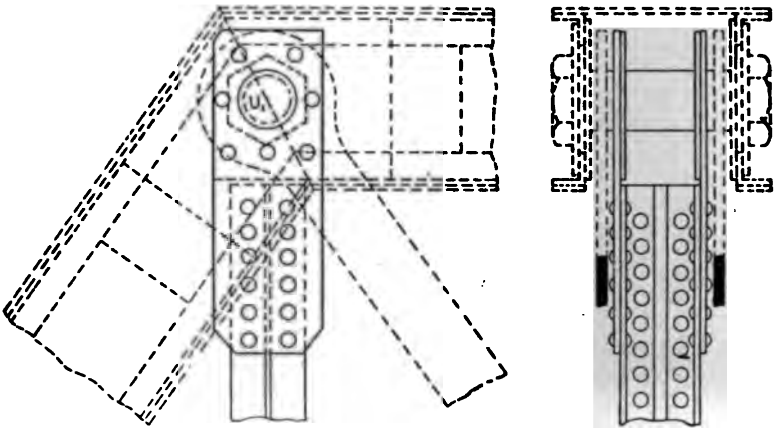


Fig. 177. Connection of Hip Vertical to Upper Chord and End-Post.

The joint between the plates and the main member will be weak in shear, the rivets tending to shear off between the ¾-inch angles and the plate, and also between the two plates themselves. As each side takes one-half of the above stress, the number of rivets required to connect the plates to the main member will be:

$$\frac{141\,600 \div 2}{7\,220} = 10 \text{ shop rivets,}$$

and the number of rivets required to connect the inner ½-inch plate to the outer one which is connected to the member itself will be:

$$\frac{141\,600 \div 4}{7\,220} = 5 \text{ shop rivets.}$$

The distance from the center of the pin to the top of the main part must be greater than one-half the diameter of the largest I-bar head—that is, 17½ ÷ 2 = say, 9 inches.

At the lower end, this member is connected to the bottom chord by means of a couple of clip angles and four or five rivets. Only sufficient rivets are required to prevent the sagging of the bottom chord, since the floor-beam is connected to the hip vertical above the lower chord, and hence no stress comes on the joint at the lower end (see Fig. 178).

The width of the plate has been assumed as 8 inches. This width is liable to be changed after the design of the intermediate posts has been made, since it will be economical to have all the intermediate floor-beams of the same length; and therefore the width of this plate will be changed so as to make the width of the hip vertical the same as the width of the intermediate posts.

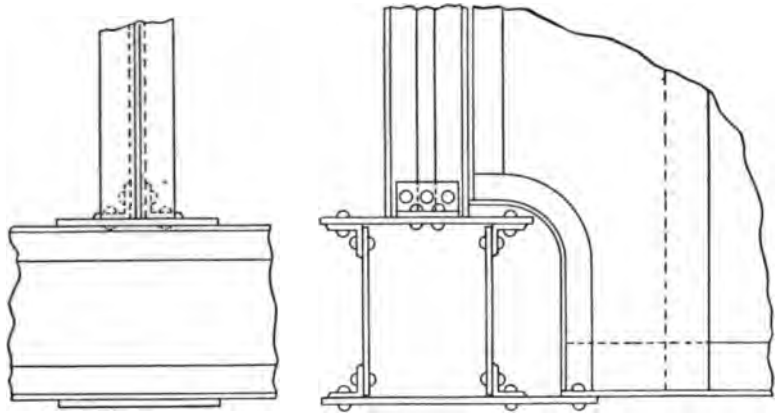


Fig. 178. Connection of Hip Vertical to Lower Chord.

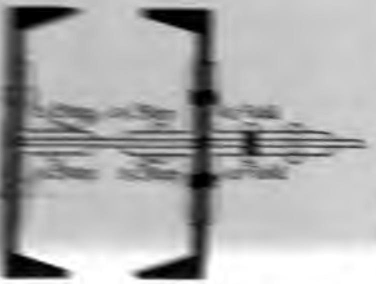
85. **The Intermediate Posts.** The post U_2L_2 must be designed to stand a total stress of 163 600 pounds. Where possible, it is economical to make the intermediate posts out of channels, as this saves a large amount of riveting. As seen by the stress sheet, the length of these posts is 30.1 feet center to center of end pins. It is usually required that $\frac{l}{r}$ must not be more than 100, and this condition requires that the least radius of gyration cannot be less than $\frac{30.1 \times 12}{100} = 3.62$.

From Carnegie Handbook, p. [illegible], it is seen that a 12-inch 30-pound channel has a radius of gyration of 4.28, and will fulfil the conditions. The area of two of these channels is 17.64 square inches. The unit allowable stress (P) is:

$$P = 16\,000 - 70\,[illegible] = \frac{[illegible]}{4.28} = 10\,090 \text{ pounds per square inch.}$$

The required area is thus determined to be $\frac{163\,600}{10\,090} = 16.2$ square inches and as this coincides very closely with the area given, these channels are efficient and will be used.

Fig. [illegible] shows the cross-section of this post. The radius of gyration which was used above was the radius of gyration of the channels about an axis perpendicular to their web. The radius of gyration of the entire section about an axis perpendicular to the web will be the same as that of one channel. In order to have the sections safe, the radius of gyration about the axis *B–B* must be equal to or greater than the other. The radius of gyration about the axis *B–B* can be increased or decreased by spacing the channels. The exact distance which will make the two rectangular radii of gyration equal may be determined by the methods of "Strength of Materials," or it may be found in columns [illegible] and [illegible] of the Carnegie Handbook, p. [illegible] [illegible] given in column [illegible] For the channels under [illegible] inches. Any [illegible] distance will tend to increase the radius of gyration about the axis *B–B* and will make the post safer about that axis.



[illegible]

[illegible] diaphragm. The web of this diaphragm cannot be less than [illegible] and the size of the angles cannot be less than 3½ [illegible] as this is the least allowed by the Specifications. The function of this diaphragm is to transfer one-half of the floor-beam reaction to the outer side of the post. The rivets which connect the angles to the diaphragm web are shop rivets, and (see design of floor-beam) must be $\frac{137\,600}{2 \times 7\,880} = 9$ in number. The rivets which connect the diaphragm angle with the outer channel of the post are also shop rivets, and are $\frac{137\,600}{2 \times 7\,220} = 10$ in number, 5 on each side. The same rivets which connect the floor-beam to the post go through the diaphragm angle on that side of the diaphragm next to the center of the bridge, and must therefore be field rivets and take the entire floor-beam reaction. These must be $\frac{137\,600}{6\,013} = 23$ in number, 12 on each side. The exact distance, back to back of the channels of the post, cannot be determined until after the top chord has been designed, since the post must slide up in the top chord and also leave room on each side for the diagonal members of the truss. The width is determined by the packing of the members at joint L_2 (see Fig. 174), and is found to be 9¼ inches. Since this is less than that required above, the post must be examined for bending about an axis parallel to the web of the channels.

According to the methods of "Mechanics" and "Strength of Materials," with the help of the Carnegie Handbook, p. 102, the moment of inertia about this axis is found to be 286.42, and the radius of gyration 3.96. The unit allowable compressive stress is then computed to be 9 580 pounds per square inch, and the required area $\frac{163\,600}{9\,580} = 17.10$ square inches, which, being less than 17.64, shows the section to be safe.

This member is connected to the top chord at its upper end by a 5-inch pin. The total stress is 163 600 pounds, and the total bearing area required is $\frac{163\,600}{24\,000} = 6.8$ square inches, or 3.4 square inches for each side (19). The total thickness of the bearing area for each side is $\frac{3.4}{5} = 0.68$ inch. The thickness of the web of a 12-inch 30-pound channel is 0.513 inch, which leaves $0.68 - 0.513 = 0.167$ inch as the thickness of the pin-plate, but it must be made ⅜ inch according to the Specifications. Fig. 180 shows the arrangement of the plates and the rivets.

The sum total of the pin-plates and the channel web is 0.888 inch, and therefore on one side the stress transferred to the pin by means of the pin-plate, which is 0.375 inch, is $\frac{1}{2} \times \frac{0.375 \times 163\,600}{0.888} = 34\,600$ pounds. This plate will tend to shear off the rivets between it and the channel web, and therefore $\frac{34\,600}{7\,220} = 5$ shop rivets are required.

The stress that is shown on the stress sheet is the stress in the post above the floor-beam. The stress in that part below the floor-beam is equal to the vertical component of the diagonal in the panel ahead of the post in question. In this case it is the vertical component of the stress in U_1L_2, and is equal to 242 000 pounds, and this requires a total bearing area of $\frac{242\,000}{24\,000} = 10.1$ square inches, and a total thickness of $\frac{10.1}{2 \times 5} = 1.01$ inches on each side, the pin being 5 inches in diameter. From this total thickness must be subtracted the thickness of the web of the channel, and this leaves $1.01 - 0.513 = 0.497$ inch as the total thickness of the pin-plates required. This shows that we must use one ½-inch plate. The total thickness of the bearing area is now $0.513 + 0.50 = 1.013$ inches.

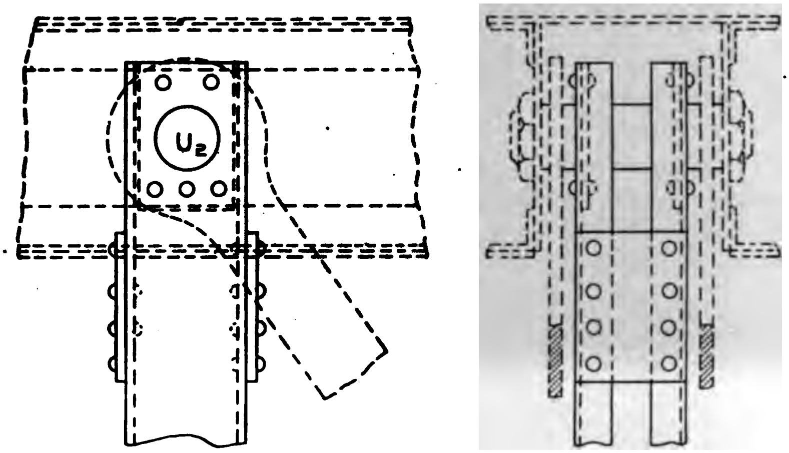


Fig. 180. Arrangement of Plates, Rivets, Pin, etc., at Connection of Intermediate Post to Top Chord.

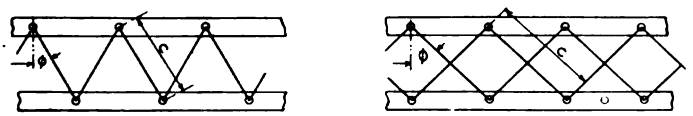
Each plate takes a total stress of $\frac{0.50}{1.013} \times \frac{242\,000}{2} = 59\,700$

pounds; and the joint being weak in shear, the number of rivets required will be $\frac{59\,700}{7\,220} = 9$ rivets in single shear. The detail will be similar to that in Fig. 180.

The distance, back to back of the channels in this post, will probably not be greater than 12 inches, and this will make the distance between rivet lines about 9 inches. According to (44), the end tie-plates must be at least 9 inches long and of course 12 inches wide. The thickness cannot be less than $\frac{9}{50} = 0.18$ inch, but they will be made ⅜ inch (36). Between the tie-plates the channels will be connected by means of lattices. The Specifications (45) require that they should not be less than 2¼ inches in width and $(1.414 \times 9)\frac{1}{40} = 0.318$ (say ⅜) inch in thickness. Table XXV gives the thickness of lacing bars for any distance between rivets.

TABLE XXV

Thickness of Lacing Bars



SINGLE LACING $\left(t = \frac{c}{40}\ ;\ \phi = 30^\circ\right)$		DOUBLE LACING $\left(t = \frac{c}{60};\ \phi = 45^\circ\right)$	
t	c	t	c
¼ in.	0 ft. 10 in.	¼ in.	1 ft. 3 in.
5/16 in.	1 ft. 0½ in.	5/16 in.	1 ft. 6¾ in.
⅜ in.	1 ft. 3 in.	⅜ in.	1 ft. 10½ in.
7/16 in.	1 ft. 5½ in.	7/16 in.	2 ft. 2¼ in.
½ in.	1 ft. 8 in.	½ in.	2 ft. 6 in.
9/16 in.	1 ft. 10½ in.	9/16 in.	2 ft. 9¾ in.
⅝ in.	2 ft. 1 in.	⅝ in.	3 ft. 1½ in.

A width of 2¼ inches is chosen above, since according to Carnegie Handbook, p. 183, a ¾-inch rivet is the largest which can be used in the channel flange.

The post U_3L_3 must be designed for a total stress of 87 000 pounds. It will be assumed that two 10-inch 20-pound channels with a radius of gyration 3.66 and an area of 5.80 square inches each will be sufficient. The length, as before, is 30.1 feet, and the unit-stress is:

$$P = 16\,000 - 70 \times \frac{30.1 \times 12}{3.66} = 9\,080 \text{ pounds.}$$

The required area is $\frac{87\,000}{9\,080} = 9.60$ square inches. Since the total area of the two channels is 11.76 square inches, and the required area is 9.6 square inches, it is seen that they do not coincide very closely. These channels, however, will be used, since the thickness of the web is the thinnest allowed by the Specifications, and the width of the channels is the smallest that can be used and still give sufficient room to make the connections with the end connection angles of the floor-beams.

The lower end of this post also has a diaphragm which must transfer half of the stress to the outer channel of the post. The sides of the diaphragm are the same as in the posts previously designed; and the number of rivets required is computed in a similar manner and found to be as indicated in Fig. 181, which shows the cross-section of this post.

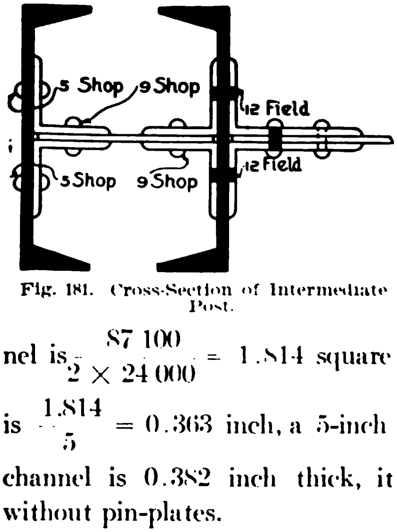


Fig. 181. Cross-Section of Intermediate Post.

At the upper end the bearing area required on one channel is $\frac{87\,100}{2 \times 24\,000} = 1.814$ square inches, and the thickness required is $\frac{1.814}{5} = 0.363$ inch, a 5-inch pin being used. As the web of the channel is 0.382 inch thick, it will give sufficient bearing area without pin-plates.

At the lower end, the vertical component of U_2L_3 is 157 500 pounds. The bearing area required on each side of the post is $\frac{157\,500}{2 \times 24\,000} = 3.28$ square inches, and the thickness is $\frac{3.28}{5} = 0.66$ inch. The thickness of the channel web being 0.382 inch leaves $0.660 - 0.382 = 0.278$ inch as the required thickness of the pin-plate;

but $\frac{3}{8}$ inch must be used, making a total thickness of 0.382 + 0.375 = 0.757 inch. The plate will carry $\frac{0.375}{0.757} \times \frac{157,500}{2} = 39,000$ pounds, and this requires $\frac{39,000}{7,220} = 6$ shop rivets in single shear.

The distance, back to back of channels, will be the same as in

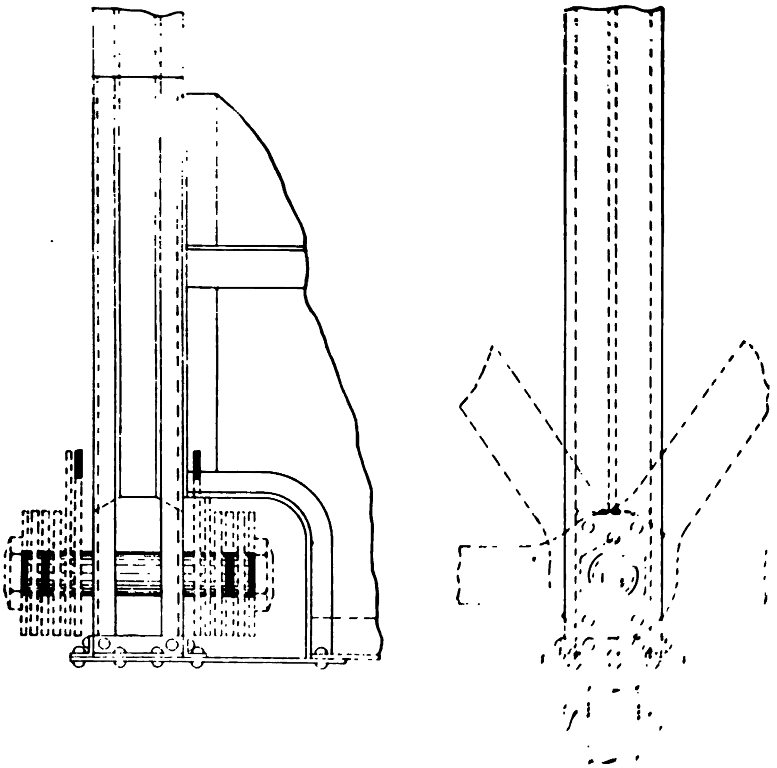


Fig. 182. End and Side Elevations Showing [illegible]

[illegible]

U_2L_2, and therefore the tie-plate [illegible]
Fig. 182 gives a detail of the [illegible]

86. The Top Chord. [illegible] may be made of two channels [illegible] This is not very good [illegible] open and lets in the [illegible] joints. It is better [illegible]

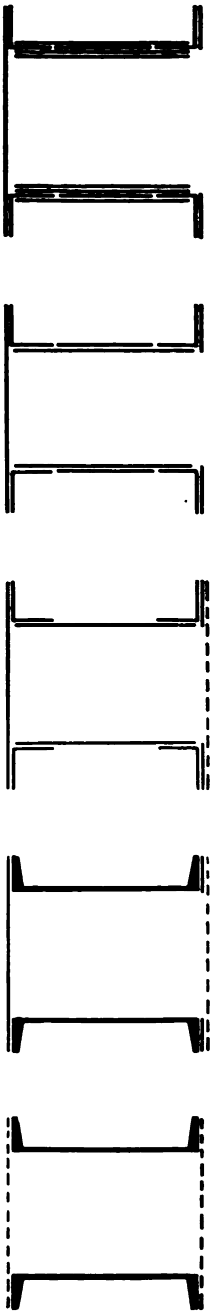


Fig. 183. Chord Sections, Showing Various Types of Construction.

an excessive section. In case of stress such as is demanded, the chords may consist of two channels and a cover-plate. In this case it is necessary to place small pieces called *flats* upon the lower flanges of the channel, in order to lower the center of gravity of the section and to bring it near the center of the web. This section makes a very economical section in that it saves much riveting. On account of channels being made only up to 15 inches in depth, the use of this section is quite limited owing to the fact that it is not deep enough to allow the I-bar heads sufficient clearance, for the I-bar heads in bridges of even ordinary span will exceed this amount.

The most common section is that which consists of two side plates, four angles, and one cover-plate. Sometimes this section has flats placed upon the lower angle in order to lower the center of gravity, as explained above. According to (33), the section should be as symmetrical as possible, and the center of gravity should lie as near the center of the web as is consistent with economy.

In case the stress is great enough to demand a heavier section than that above described, additional plates are added upon the sides of the original plates, and heavier and larger cover-plates and angles are used. Fig. 183 shows different types of chord sections.

In addition to the cover-plate being designed to withstand the total stress, close attention must be paid to (42). This clause has been inserted on account of practical considerations, since it has been found out that if plates are made much thinner than the proportions here required, they will crumple up and fail long before the allowable unit of stress as computed from the formula has been reached. In some cases—especially where the stress is light—the proportions laid down in (42) and (36) will govern the design of the section, instead of the required net area as determined by the formula for the allowable unit compressive stress.

The design of the first section of the top chord will now be made. Here, as in the case of the first sections of the lower chord, the diameter of the head of the greatest I-bar determines the width of the plates in the section. The head of the 7-inch I-bar which constitutes the member U_1L_2 is 17½ inches, and, allowing a clearance of ½ inch on either side of the head, the total depth inside the chord should be 18½ inches. As in the case of the lower chord, plates 18 inches wide may be used.

The size of the angles to be chosen is a matter of judgment. Usually any size should be chosen at first, and the preliminary design will indicate at once what size should have been taken. For this case, 3½ by 3½ by ⅜-inch will be assumed at first.

For sections of this character, the radius of gyration is approximately equal to $0.4h$, in which h is the height, or rather the width, of the side plate. The approximate radius of gyration is $r = 0.4 \times 18 = 7.2$ inches, and the length is equal to one panel length, or 21 feet. The allowable unit of stress (16) is:

$$P = 16\,000 - 70 \times \frac{21 \times 12}{7.2} = 13\,550 \text{ pounds.}$$

The required area is $\frac{449\,500}{13\,550} = 33.2$ square inches. The correct proportion for sections of this character is that 0.4 of the total area should be taken up by the web. The area of the web would then be $0.4 \times 33.2 = 13.28$ square inches, and the thickness would be $\frac{13.28}{2 \times 18} = 0.37$ inch. According to this, a ⅜-inch plate should be used, but (42) requires that it shall be $\frac{14.5}{30} = 0.483$ inch or thicker. Therefore an 18 by ½-inch plate must be used for the web.

The correct proportion for sections of this character is that the width between plates should be about ⅞ the width of the side plates.

s will give the required width between plates equal to $\frac{7}{8} \times 18 =$ 15.75 inches. The cover-plate (42) must not be thinner than $\frac{1}{40}$ he distance between the connecting rivet lines. The rivet lines are, in this case, $15.75 + 2 \times 2 = 19.75$ inches apart, and therefore the thickness of the cover-plate cannot be less than $\frac{19.75}{40} = 0.494$ inch. The cover-plate will therefore be taken as ½ inch thick. The width of the cover-plate (see Fig. 184) must be about $15.75 + 2 \times 3\frac{1}{2} + \frac{1}{2} = 23\frac{1}{4}$ inches (say 23 inches). The cover-plate will be taken 23 by ½-inch.

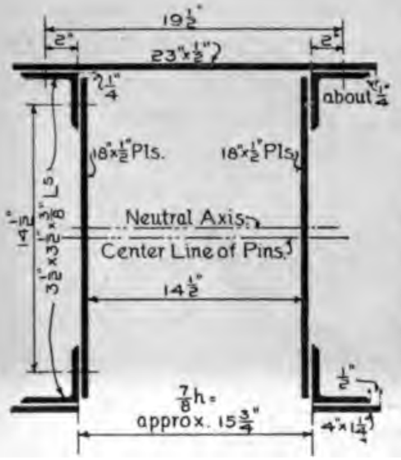


Fig. 184. Section of Top Chord.

The center line of pins will be taken at the center line of the web, and the center of gravity of the section will be assumed as ½ inch above this. In order that the center of gravity may be near that assumed, the moment of the cover-plate about the assumed center of gravity axis should be about equal to the moment of the flats about the same axis. The moment of the cover-plate about the assumed axis is:

$$23 \times \tfrac{1}{2}\,(9.0 - 0.5 + 0.25 + 0.25) = \frac{23 \times 9}{2};$$

and the moment of the flats about the same axis is:

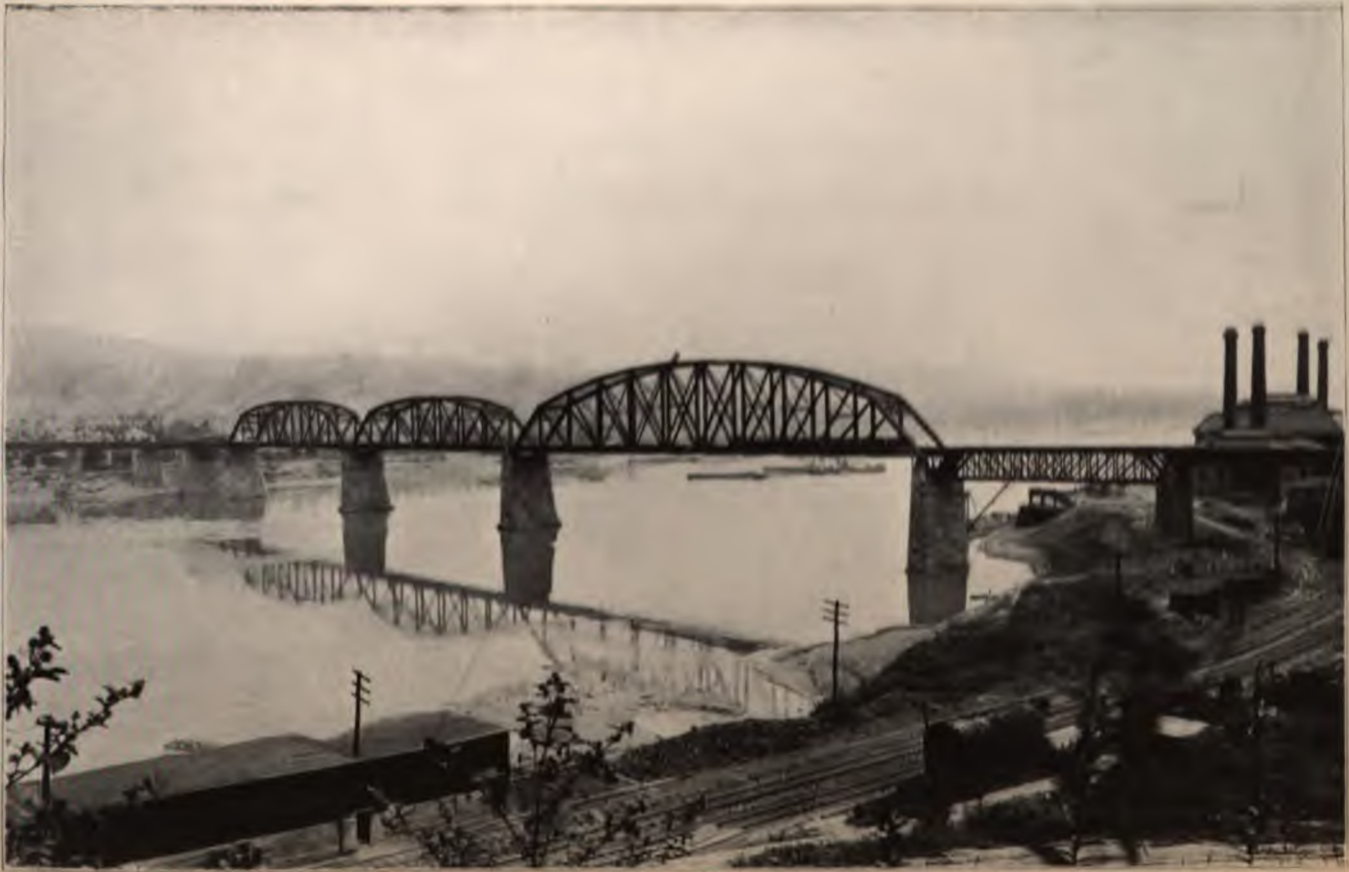
$$A\,(9.0 + 0.5 + 0.25 + 0.5) = 10.25\,A,$$

in which A is the area in square inches of both of the flats. Equating these two expressions, and solving for A, there results:

$$A = \frac{23 \times 9}{2 \times 10.25} = 10.1 \text{ square inches.}$$

Assuming the flats to be 4 inches wide, the thickness on each side will be 1.25 inches. As this is too thick to punch, the flats on each side will be composed of two 4 by ⅝-inch plates.

The total area is:



BRIDGE OVER ALLEGHENY RIVER AT BRILLIANT, PENNSYLVANIA

One cover-plate = 23 × ½	= 11.5	sq. in.
Two web plates = 2 × 18 × ½	= 18.0	" "
Two flats 4 × 1¼	= 10.0	" "
Total	39.5	sq. in.

But the required area is 32.2 square inches, which is considerably less than the area above given, and which does not include the angles and hence we can use the smallest size angles, which are those previously assumed. The area of each of these angles is 2.48 square inches, thus making the total area of the section 39.5 + 4 × 2.48 = 49.42 square inches. This is considerably in excess of the area as required according to the formula for compression; but it is the least allowed by the Specifications. Note that this is the case where (42), instead of the formula for compressive stress, is the ruling factor in the determination of the section.

The center of gravity of the approximate section must now be determined, the moment of inertia and the radius of gyration about the neutral axis must be computed, and the required area must be determined by using this radius of gyration as computed. If the required area as determined with the actual radius of gyration is less than the approximate area, then the thickness of the angles or the plates must be increased and the section then examined for its radius of gyration and required area. If the area is sufficient, the section is used; if not, another recomputation is in order.

In the determination of the center of gravity of the section, the moment is taken about the top of the cover-plate. The moments are computed as follows:

Cover-plate (23 × ½) × ¼	2.88
Webs 2 (18 × ½) × (9 + ¾)	175.60
Top angles 2 (2.48) × (1.01 + ½)	7.50
Lower angles 2 (2.48) × (½ + ¼ + 18 + ¼ − 1.01)	89.30
Flats 2 (4 × 1¼) × 19⅝	196.25
Total	471.53

The center of gravity is now found to be $\dfrac{471.53}{39.50 + 4 \times 2.48} = 9.55$ inches from the top of the cover-plate. The distance from the top of the cover-plate to the middle line of the web is 9 + ¼ + ½ = 9.75 inches, and this leaves a distance of 9.75 − 9.55 = 0.2 inch from the center line of the web to the neutral axis. This distance is generally represented by the letter e, and it is known as the *eccentricity of the section.*

The moment of inertia about this axis must now be computed. The relation used is that the moment of inertia about any axis is equal to the moment of inertia about some other axis, *plus* the product of the square of the distance between the two axes by the area of the section whose moment of inertia is desired. The moments of inertia of the various parts of the section (see "Steel Construction," Part IV, pp. 292 and 293) are computed and are as follows:

Cover-plates	955.26
Webs.	486.72
Top angles.	325.74
Lower angles	359.74
Flats.	1 017.37
Total.	3 184.83

The radius of gyration is equal to the square root of the quotient obtained by dividing the moment of inertia by the area. It is

$$r = \sqrt{\frac{3\,184.83}{49.42}} = 8.04.$$

Using this value of the radius of gyration in the formula for the compressive stress, there is obtained 13 800 pounds as the unit allowable stress in compression, and this requires an area of $\frac{449\,500}{13\,800} = 32.5$ square inches. Since this is considerably less than the actual area of the section, the section will not be changed but will be taken as first assumed.

In order that the section should be safe about both axes, the moment of inertia about the axis perpendicular to the cover-plate should be equal to or greater than that as above computed. By computing the moment of inertia about the axis perpendicular to the cover-plate, it is found to be 3 256.3, which gives a radius of gyration of 8.11; and since both of these are greater than those first computed, it is seen that the section is safer about the axis perpendicular to the cover-plate than it is about an axis perpendicular to the web plates.

There are small stresses in this member due to its own weight and to the fact that the pins are not placed directly upon the neutral axis (see "Strength of Materials," p. 82). These stresses are seldom more than 1 000 pounds per square inch in the extreme fibre; and since the section has such an excess of area, they will not be computed, as it is evident that there is sufficient strength in the member to withstand them.

The section just designed is that for the top chord having the greatest stress; and since this is the minimum section allowed by the Specifications, it must be used in all the sections of the top chord.

The section as finally designed is:

One cover plate, 23 by ½ inch;
Two webs, 18 by ½ inch;
Four angles, 3½ by 3½ by ⅜-inch
Four flats, 4 by ⅝-inch.

A pin 6¼ inches in diameter will be used at the point U_1. The stress in the member U_1U_2 is 378 200 pounds, and the bearing area required is $\frac{378\,200}{24\,000} = 15.75$ square inches, or 7.875 for each side. This makes a total required thickness of $\frac{7.875}{6.25} = 1.265$ inches for one side. Since the thickness of the web plate is ½ inch, it will be necessary to provide pin-plates whose total thickness must be $1.265 - 0.5 = 0.765$ inch. Two ⅜-inch plates will give a thickness of 0.75 inch; and since this is less than the required thickness by an amount not over 2½ per cent, they may be used. The total thickness of the bearing area is now 1.265 inches. The stress transferred to the two ⅜-inch plates is:

$$s = \frac{2 \times \frac{3}{8}}{1.25} \times 189\,100 = 113\,500 \text{ pounds.}$$

The rivets required to keep the outer plate from shearing off the other are $\frac{113\,500}{2 \times 7\,220} = 8$ shop rivets, and the rivets required to keep both of the ⅜-inch plates from shearing off the web of the chord section are $\frac{113\,500}{7\,220} = 16$ shop rivets in single shear. The bearing of a ⅞-inch shop rivet on a ½-inch plate is 10 500 pounds, and therefore the number of rivets required to keep these pin-plates from tearing the rivets out of the ½-inch web plates is $\frac{113\,500}{10\,500} = 11$ shop rivets in bearing. Fig. 185 shows the detail of this end of the top chord section. The pin-plates should extend well back on the member, and at least one pin-plate should go over the angle, and enough rivets, as computed above, should go through the angles and this pin-plate. Experiments on full-sized bridge members go to show that unless the pin-plates cover the angles and extend well down on the member, the member will fail before the unit-stress reaches that value computed by the formula for compression.

Since the ends of the chord are milled at the splices, and therefore butt up against each other and allow the stress to be transmitted directly, only sufficient rivets need be placed in the splice to keep the top chord sections in line (55).

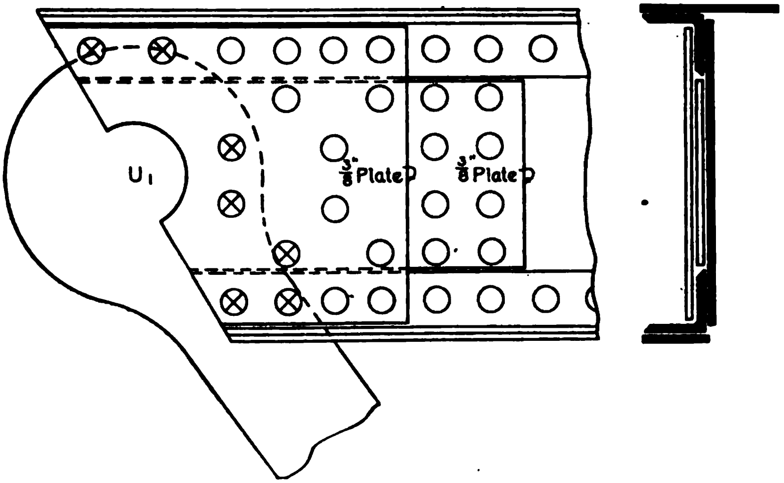


Fig. 185. Detail of Top Chord Section at Point U_1.

At the point U_2, it is not necessary to put in a pin-plate to take the stress in the upper chord; but it is only necessary to provide a pin-plate to take up the difference in stress between the two chord sections. This difference in stress is equal to the horizontal component of the maximum stress in the member U_2L_3. This is 110 000 pounds, and the area required on each side for bearing is 2.3 square inches; and as a 5-inch pin is used here, the thickness of the bearing area is $\frac{2.3}{5} = 0.46$ inch. As this thickness is less than the thickness of the web plate, no pin-plates will be required.

At the point U_3, a bearing area will be required to withstand the horizontal component of the member U_3L_4. This is 56 300, and the bearing area required on each side is $\frac{56\,300}{24\,000 \times 2} = 1.18$ inches. The required thickness of the bearing area is $\frac{1.18}{5} = 0.24$ inch, as a 5-inch pin is used here also. As this thickness is less than the thickness of the web plate, no pin-plate will be required.

The under parts of these members must be stiffened by tie or batten plates, and these plates (44) must be equal in length to the distance between rivet lines. This is $19\frac{1}{2}$ inches. They will be made 20 inches long and 23 inches wide. The thickness of these plates (44) must be $\frac{19.5}{50} = 0.39$ inch (say $\frac{7}{16}$ inch). The size of the tie-plates will then be 20 in. by $\frac{7}{16}$ in. by 1 ft. 11 in.

Since the distance between the rivet lines is greater than 15 inches, double latticing must be used (47); and according to Table XXV the lacing must be $\frac{1}{2}$ inch thick; also, according to (45), it must be $2\frac{1}{2}$ inches wide, as the rivets used are $\frac{7}{8}$ inch in diameter. The lattices will then be $2\frac{1}{2}$ by $\frac{1}{2}$-in.

87. **The End-Post.** Since the minimum section as chosen for the top chord is about 50 per cent in excess of that required by the compression formula, it will be assumed to be sufficient for the section of the end-post, and it will now be investigated to see if it is safe.

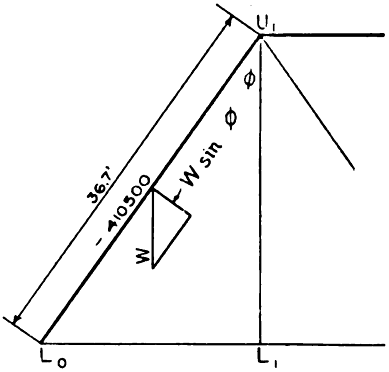


Fig. 186. Calculation of End-Post.

In addition to the stress due to direct compression, the end-post is stressed by its own weight, by eccentric loading due to the pin being in the center of the web instead of at the center of gravity of the section, and to a bending moment at the place where the portal brace joins it. This is due to the bending action of the wind on the top chord. These different stresses will now be computed; and since the post is in all cases stressed by a combination of bending and compressive stresses, this fact should be considered in the design. In determining the stress in the end-post due to its own weight, the entire weight must not be used in computing the bending action, but only that component of it which is perpendicular to the end-post. The length of the end-post is readily computed, and is as shown in Fig. 186. The general formula for accurately computing stresses due to bending when the member is also subjected to compression, is:

$$S = -\frac{My_1}{I - \frac{Pl^2}{10E}},$$

in which,

S = Stress in pounds per square inch in the extreme upper fibre of the beam;
M = Exterior moment causing the stress, and is considered positive if it bends the beam downward, and negative if it bends the beam upward;
y_1 = Distance from the neutral axis to the extreme upper fibre;
I = Moment of inertia of the section;
P = Direct compressive stress, in pounds;
l = Total length, in inches;
E = Modulus of elasticity of steel, which is usually taken as 28 000 000 pounds per square inch.

In this case the force causing the bending is that component of the weight perpendicular to the end-post. This is $Wl \sin\phi$, in which W is the weight of the steel in the end-post; and this is computed and is as follows:

Cover-plate	1 435 lbs.
Web plates	2 245 "
Angles	1 250 "
Flats	1 245 "
	6 175 lbs.
Add 25 per cent for details	1 544 "
Total	7 719 lbs.

Substituting in the above formula the various values, there results:

$$S = -\frac{\frac{1}{8} \times 7\,719 \times 36.7 \times 0.572 \times 12 \times 9.55}{3\,185 - \frac{410\,500 \times (36.7 \times 12)^2}{10 \times 28\,000\,000}}$$

= 800 pounds per square inch compression in the upper fibre due to bending.

In the above equation, the stress in the member is 410 500 pounds; the distance y_1 is the distance from the neutral axis to the top of the cover-plate, and the coefficient of elasticity of steel is taken as 28 000 000.

In computing the stress due to the eccentric loading, the moment is equal to the product of the total stress in the member by the distance from the neutral axis to the center of gravity axis causing a negative moment. Substituting in the above formula for combined stresses, there results:

$$S = -\frac{-410\,500 \times 0.2 \times 9.55}{3\,185 - \dfrac{410\,500 \times (36.7 \times 12)^2}{10 \times 28\,000\,000}}$$

= 270 pounds per square inch tension in the upper fibre.

In order to find the compression in the lower fibre, it is only necessary to notice that the stresses are proportional to the distances from the neutral axis. Accordingly (see Fig. 187), the stress in the lower fibre due to the weight is 895 pounds tension, and the stress in the lower fibre due to the eccentric loading is 302 pounds compression.

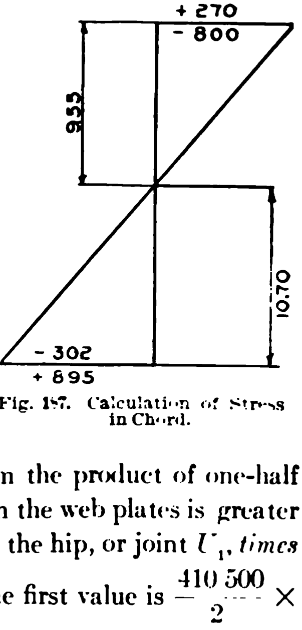


Fig. 187. Calculation of Stress in Chord.

Before computing the stress due to the bending moment caused by the wind on the upper chord, it is necessary to investigate the post to see if it is fixed or hinged at its lower end. This is very important, since, if the post is found to be hinged, the bending moment will be one-half of that which will occur when the post is not hinged.

An end-post is considered hinged when the product of one-half of the total stress *times* the distance between the web plates is greater than the product of the wind load acting at the hip, or joint U_1, *times* the length of the end-post. In this case the first value is $\frac{410\,500}{2} \times 15 = 3\,075\,000$; and the product of the latter (see Article 29) is $12\,600 \times 36.7 \times 12 = 5\,550\,000$. Since the latter is greater than the former, the post is hinged, and the bending moment at the foot of the portal strut, which joins the end-post 28.2 feet from the end, is $6\,300 \times 28.2 \times 12 = 2\,130\,000$ pound-inches. The stress in the extreme fibre due to this bending moment is:

$$S = \frac{2\,130\,000 \times 11.5}{3\,256.3 - \dfrac{410\,500 \times (36.7 \times 12)^2}{10 \times 28\,000\,000}}.$$

$= 8\,250$ pounds per square inch tension or compression.

In computing this stress due to the wind moment, care must be taken to take y_1 equal to one-half the width of the cover-plate, and to take the moment of inertia as that about the axis perpendicular to the cover-plate.

In computing the total stress on the extreme fibre, it must be noted that the stresses due to weight and eccentric loading do not stress the same extreme fibres as the stress due to wind, the former stressing the extreme fibres on the top and bottom of the post, while the latter stresses those on the inner and outer sides. The total direct unit-stress is $\frac{410\,500}{49.42} = 8\,310$ pounds per square inch; and this, added to the 8 250 pounds per square inch due to the wind, gives a total of 16 560 pounds per square inch on the extreme fibre only.

The allowable unit-stress is $16\,000 - 70 \times \frac{36.7 \times 12}{8.11} = 12\,200$ pounds per square inch when wind is not taken into account, and (23) is $1\frac{1}{4} \times 12\,200 = 15\,250$ pounds per square inch when the wind is taken into account. The difference between this and the actual stress is $16\,560 - 15\,250 = 1\,310$ pounds per square inch, which shows that the section is not strong enough. The section can be increased by widening the cover-plate or by making the plates thicker; but as this excess is due to wind only, the section being amply sufficient under the other stresses, and is fixed to some extent by the floor-beam connection, no change will be made.

The pin at each end of the end-post will be the same—namely, $6\frac{1}{4}$ inches in diameter—and therefore the pin-plates will be the same at each end. The total stress in the post is 410 500 pounds, which makes a required bearing area of $\frac{410\,500}{24\,000} = 17.2$ square inches for both sides, or 8.6 square inches for one side, and the total required thickness of $\frac{8.6}{6.25} = 1.375$ square inches for one side. Since the thickness of the web plates is $\frac{1}{2}$ inch, this leaves a remainder of $1.375 - 0.5 = 0.875$ inch for the thickness of the pin-plates. One plate $\frac{3}{8}$ inch thick and one plate $\frac{1}{2}$ inch thick will be used.

The proportion of the total stress which is taken by the $\frac{3}{8}$-inch plate is $\frac{0.375}{1.375} \times \frac{410\,500}{2} = 56\,000$ pounds; and that taken by the ½-inch plate is $\frac{0.50}{1.375} \times 205\,250 = 74\,600$ pounds. The number of rivets required to transfer the stress from the ⅜-inch plate to the ½-inch plate is $\frac{56\,000}{7\,220} = 8$ shop rivets in single shear; and the number of rivets required to transfer the stress from both pin-plates to the web is $\frac{56\,000 + 74\,600}{7\,220} = 18$ shop rivets in single shear. As in the case of the top chord, one pin-plate should extend over the angle, and the number of rivets required in that pin-plate should go through the pin-plate and the angles (see Fig. 188). The ⅜-in. hinge plate is used for erection purposes, and is not considered as a pin-plate. It is omitted at L_0.

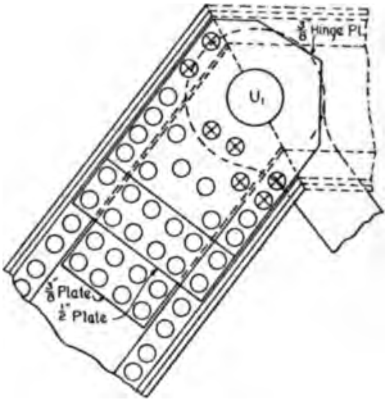


Fig. 188. Plates and Riveting at Upper End of End-Post.

Since this section is the same as that of the top chord, the tie-plates and the lattice bars must be the same size.

88. **The Pins.** The design of the pins requires a simple but quite lengthy computation. Simple Pratt railroad trusses for single-track bridges usually have the same arrangement of tension and compression members; that is, the same tension members occupy relatively the same positions with respect to the compression members. Also, while theoretically a different sized pin will be required at every joint, it is not customary to make them so. In practice the pins at the joints U_1 and L_0 are made of the same diameter, and those at the remainder of the joints are also made in diameter equal to each other but different from those at U_1 and L_0, the pins at U_1 and L_0 usually being larger in diameter. On account of the above conditions and facts, it is unnecessary to design the pins in spans under 200 feet, since usually they are the same for any given span and loading. Table XXVI gives the diameters of pins for spans of 100 up to 200 feet for loading E 50.

TABLE XXVI

Pins for Single-Track Bridges

Loading E 50

Span	Diameter of Pin	
	U_1 and L_0	All Others
100 feet	4½ inches	4 inches
125 "	5½ "	5 "
150 "	6½ "	5¼ "
175 "	6¾ "	5¾ "
200 "	7 "	6 "

For E 40 loading, decrease the above values by ¼ inch; for E 30 loading, decrease them by ½ inch. The diameter of pins for spans not given in the table can be interpolated from the given values. No pin should be less than 3½ inches in diameter.

The span of this bridge is 147 (say 150) feet, and the diameter of the pins at U_1 and L_0 is $6\frac{1}{2} - \frac{1}{4} = 6\frac{1}{4}$ inches; and the diameter of the pins at the other panel points is $5\frac{1}{4} - \frac{1}{4} = 5$ inches. It should be noted that no pin is required at point L_1, as the two memrs which join here are built-up members and are riveted together.

The above table is for single-track bridges only. The diameters of pins for double-track bridges are given in Table XXVII. These values are for E 50 loading; and for E 40 and E 30 loading, deductions must be made as required in the case of Table XXVI.

TABLE XXVII

Pins for Double-Track Bridges

Loading E 50

Span	Diameter of Pin	
	U_1 and L_0	All Others
100 feet	6 inches	5¼ inches
125 "	8 "	6½ "
150 "	9 "	7½ "
175 "	9¼ "	8¼ "
200 "	9½ "	8½ "

No pin in a double-track bridge should be less than 4½ inches in diameter.

Pins for highway bridges are usually much less in diameter than those for railway bridges, except in the case of first-class trusses for heavy interurban traffic or for city bridges carrying paved streets, where they should be taken equal to those given for E 30 loading. Table XXVIII gives the diameters of pins for different length spans of simple highway bridges designed for 16-ton road-rollers or farm wagons and 100 pounds per square foot of roadway.

TABLE XXVIII

Pins for Country Highway Bridges

Span	Diameter of Pin	
	U_1 and Lower Chord	Upper Chord
50 feet	2½ inches	2 inches
100 "	3 "	2½ "
150 "	3½ "	2¾ "
200 "	4 "	3 "

89. **The Portal.** In order to have a clearance of 21 feet (2) above the top of rail, it is necessary that the portal be as shown in Fig. 189. The stresses are found by methods of Article 54, Part I, the wind load being computed according to (40). It must be remembered that the column is hinged.

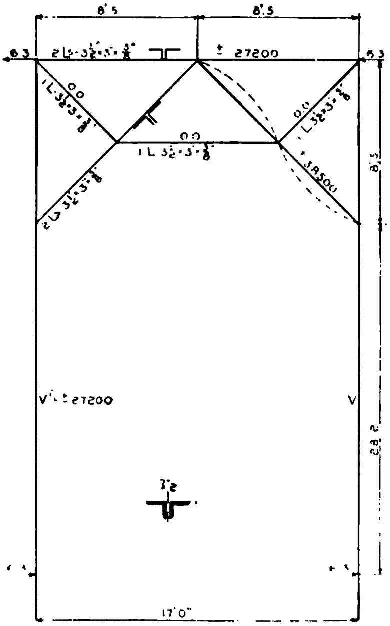


Fig. 189. Portal Dimension and Stress Diagram.

In case the members of the portal braces bend about one axis, their length will be equal to the distance from one end to the other. In case they bend about the other axis as indicated by the broken line in Fig. 189, their length will be one-half of what it was in the first case.

The portal struts or diagonals will be designed first. Their length is 8.5 × 1.414 = 12 feet, or 144 inches. This is the total length. Although the Specifications do not mention it, the ratio of the length to the radius of gyration should not exceed 120. This means that the radius of gyration in this case should be greater than $\frac{144}{120} = 1.2$. The section of the strut will be composed of two angles placed back to back.

Two angles 3½ by 3 by ⅜-inch, with an area of 4.6 square inches and r_2 equal to 1.72—see Carnegie Handbook, p. 146, and (72)—will be assumed to be sufficient to take the stress, and they must now be examined to see if the assumption is correct.

The allowable unit-stress (23) is 25 per cent greater than in the case of live or dead loads. This makes the unit-stress as computed from the formula:

$$P = \left(16\,000 - 70 \times \frac{144}{1.72}\right) 1\tfrac{1}{4} = 12\,680 \text{ pounds per square inch.}$$

The required area is $\frac{38\,500}{12\,680} = 3.05$ square inches; and since this is less than the given area, the angle will be amply sufficient. The required area is over one square inch less than the given area, but this angle must be used, since it is the smallest angle allowed by the Specifications. Note that unequal legged angles should be used, as this will make the radius of gyration about one axis larger than about the other; and this will prove economical, since, when one axis is considered, the length of the member is greater than when the other is considered.

The above angle should also be examined for tension, it being considered that one rivet-hole is taken out of the section of each angle. The net section of the two angles will now be $4.60 - 2\left(\frac{7}{8} + \frac{1}{8}\right) \times \frac{3}{8} = 3.85$ square inches; and the area required for tension is $\frac{38\,500}{16\,000 \times 1.25} = 1.93$ square inches, which shows that the angle is amply sufficient. It should be noted that these Specifications do not require that only one leg of the angle shall be efficient unless both legs are connected. In case this strut had been designed according to Cooper's Specifications, two angles 5 by 3 by ½-inch would have been required, and the 5-inch leg would have been placed vertically and the angle connected by this leg alone. While it is not within the province of this work to discuss the question of connecting angles by one or by both legs, yet it might be said that tests made on angles connected with one leg only, seem to indicate that the ultimate strength in tension is about 60 per cent of that obtained from the same angle when tested with both legs connected.

While according to (20) the alternate strains in the wind bracing do not have to be considered, since they do not occur very closely together, yet in framing the connections it is required that the sum of both positive and negative stresses shall be added. In this case the stress for which the connections must be designed is 2 × 38 500 = 77 000. It must be remembered that in this case also, the unit-stresses are increased 25 per cent over those allowed for live and dead loads.

The number of rivets required in the end connections will be governed by bearing in the connection plates, and these plates are usually made $\frac{3}{8}$-inch thick. The number of rivets required is $\frac{77\,000}{7\,880 \times 1\frac{1}{4}} = 8$ shop rivets, or $\frac{77\,000}{6\,560 \times 1\frac{1}{4}} = 10$ field rivets.

The portal bracing is riveted up in the shop and brought to the bridge site, where it is connected to the trusses by field-riveted connections at its end. Therefore the end of the portal struts which connect with the top piece will have 8 shop rivets, and the other end which connects with the end-post will have 10 field rivets. Since the angles are small, all the above rivets must go in one line, and this will cause the connection plate to be quite large. It will probably be better to connect both legs of the angle by means of clip angles and thus reduce the size of the connection plates.

The top part of the portal bracing will consist of two angles. Two angles $3\frac{1}{2}$ by 3 by $\frac{3}{8}$-inch will be assumed and examined to determine if the area is sufficient. The length of this strut is the distance center to center of trusses, and is equal to 17 × 12 = 204 inches. The least radius of gyration is therefore $\frac{204}{120} = 1.70$. The radius of gyration of the two angles assumed is 1.72 when referred to an axis parallel to the shorter leg when the two angles are placed back to back and one-half inch apart. The unit-stress is now computed:

$$P = \left(16\,000 - 70 \times \frac{204}{1.72}\right) 1\tfrac{1}{4} = 9\,625 \text{ pounds per square inch.}$$

The required area is $\frac{27\,200}{9\,625} = 2.825$ square inches. This is considerably less than the area given by the two angles; but as these are the minimum angles allowable, they must be used. Since the stress in this case is less than in the previous case, and since the angles used are the same, it is evident that these angles are safe in tension.

The number of rivets is determined by the bearing in the ⅜-inch connection plates, and is:

$$\frac{2 \times 27\,200}{7\,880 \times 1.25} = \text{ 6 shop rivets, and}$$

$$\frac{2 \times 27\,200}{6\,560 \times 1.25} = \text{ 10 field rivets.}$$

As in the case of the lateral strut, this member should be connected by both legs of the angle in order to reduce the size of the connection plates. Fig. 190 gives the details of the portal bracing and its method of connection to the end-post. The full circles represent shop rivets, and the blackened circles represent field rivets. Some engineers connect the portal bracing to the top cover-plate of the end-post. This produces an excessive eccentricity in the end-post and is bad practice.

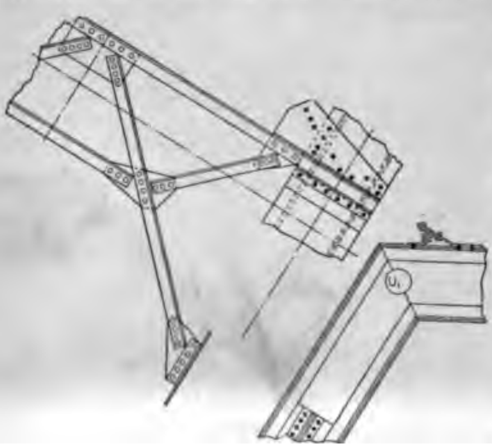


Fig. 190. Details of Portal Bracing and Connection to End-Post.

Those members of the portal bracing which do not take any stress will be made of single angles, and the size of these angles will be taken 3½ by 3 by ⅜-inch.

90. **The Transverse Bracing.** This bracing will be the same general style as the portal bracing, except that the top member will consist of two angles placed at a distance apart equal to the depth of the top chord, and these angles will be joined together by lacing. As in the case of portal bracing, those members which do not take stress will be made of one angle 3½ by 3 by ⅜-inch.

The general outline is shown in Fig. 191, and the stresses are computed from (10) and by the methods of Article 54, Part I. In designing this top member, the top angle only is supposed to take the stress. The length in this case is 204 inches. Two 3½ by 3 by ⅜-inch angles will be assumed as sufficient and will be examined. These angles ι total area of 4.60 square inches. In examining these it will ınd that they are amply sufficient, in fact so much so that it will tter to see if one single angle at the top will not be better. ding to the length, the smallest radius of gyration which can be 's 1.7. In looking over the tables of angles, it is seen that the

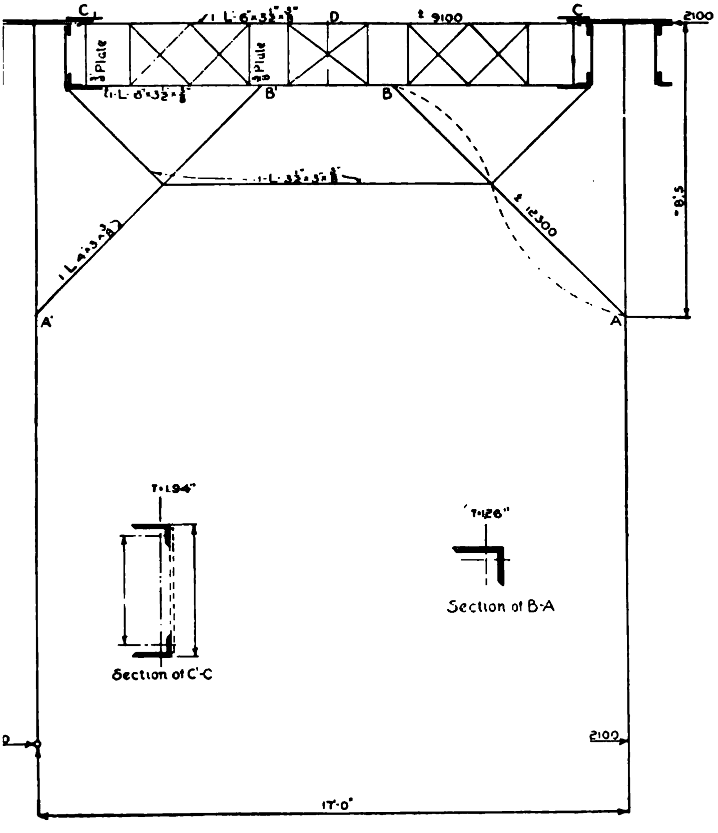


Fig. 191. General Outline of Transverse Bracing.

ngle to fulfil this condition is a 6 by $3\frac{1}{2}$ by $\frac{3}{8}$-inch, and it has a of gyration of 1.94. The allowable unit-stress is computed ows:

$$' = \left(16\,000 - 70 \times \frac{204}{1.94}\right)1\frac{1}{4} = 10\,780 \text{ pounds per square inch;}$$

e required area is $\frac{9\,100}{10\,780} = 0.85$ square inch. This is considerably smaller than the area of the angle, which is 3.97 square inches; but since this is the smallest possible angle which will fulfil the conditions of the Specifications, and since it is much smaller than the two angles as first assumed, it will be used. Fig. 192 gives a cross-section of this member. Since this angle is joined to the cover-plate by one leg, the joints will be weak in single shear, and the number of rivets required will be:

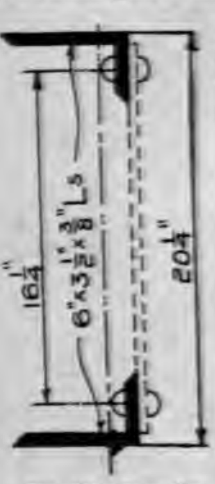


Fig. 192. Cross-Section of Top Member of Transverse Bracing.

$$\frac{2 \times 9\,100}{7\,220 \times 1\frac{1}{4}} = 2 \text{ shop rivets, or}$$

$$\frac{2 \times 9\,100}{6\,013 \times 1\frac{1}{4}} = 3 \text{ field rivets.}$$

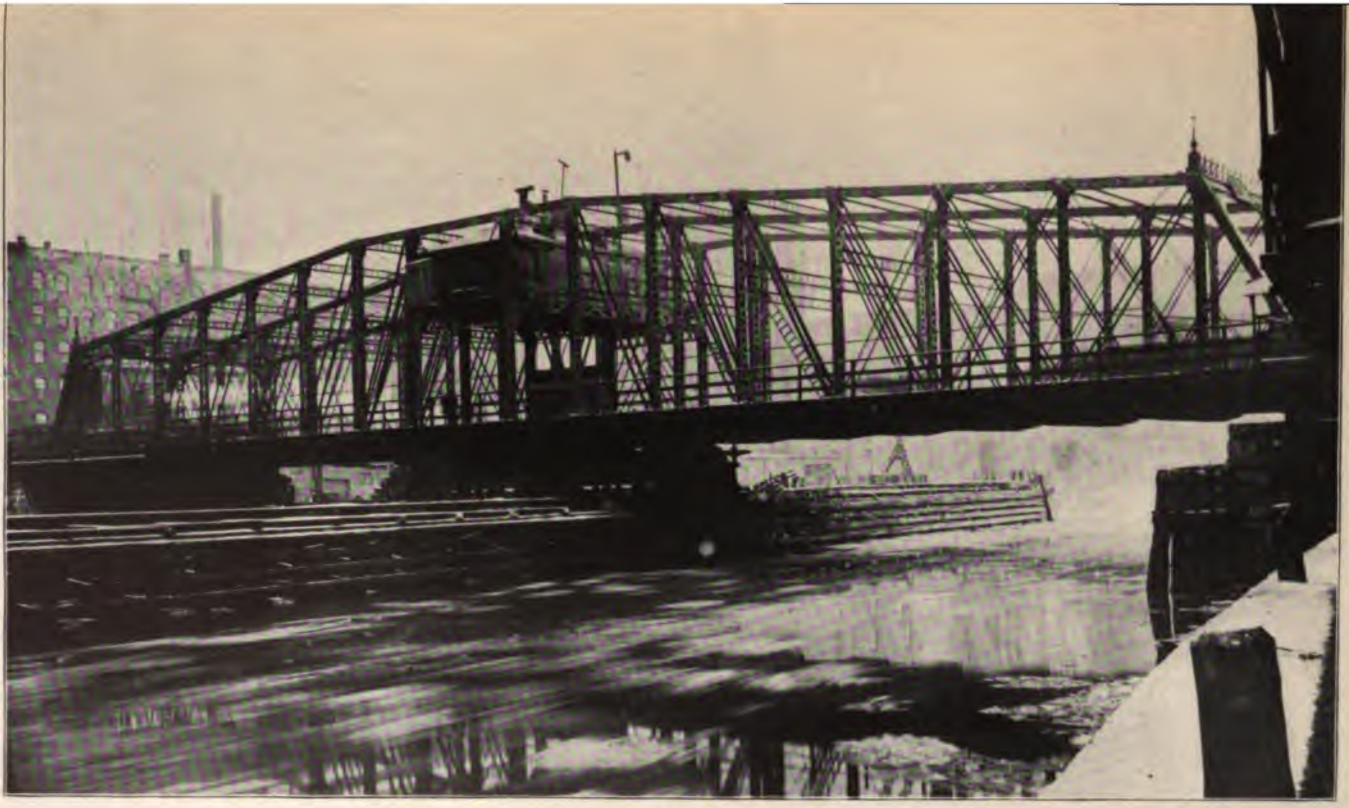
According to (45), the width of the latticing must be 2¼ inches; and according to Table XXV, the thickness must be $\frac{7}{16}$ inch, the distance c being 1 foot 11 inches.

The length of the knee-bracing is 144 inches; but on account of the small stress, one angle will be used. One 4 by 3 by ⅜-inch angle, with an area of 2.48 square inches and a radius of gyration 1.26, will be assumed as sufficient. The radius of gyration is greater than the minimum allowable, which is 1.2. The allowable unit-stress is:

$$P = \left(16\,000 - 70 \times \frac{144}{1.26}\right) 1\frac{1}{4} = 10\,000 \text{ pounds per square inch.}$$

The required area is $\frac{12\,300}{10\,000} = 1.23$ square inches. The required area is much less than the given area; but this angle must be used, since it is the only one allowed on account of its radius of gyration. Two of the minimum sized angles might have been used; but their total area, 4.60 square inches, is much in excess of that of the angle used.

This angle must be examined for tension. The net area is $2.48 - (\frac{7}{8} + \frac{1}{8}) \times \frac{3}{8} = 2.1$ square inches. The required net area in tension is $\frac{12\,300}{16\,000 \times 1\frac{1}{4}} = 0.615$ square inch, which shows this angle to be amply sufficient.



RUSH STREET SWING BRIDGE OVER CHICAGO RIVER, CHICAGO, ILLINOIS

A type of structure which, owing to its obstruction to navigation, swinging on a pier located in mid-stream, is fast giving way to the more modern type of the rolling lift bridge.

The number of rivets required will be governed by the shear, since the angle is connected by one leg only; and it is:

$$\frac{2 \times 12\,300}{7\,220 \times 1\frac{1}{4}} = 3 \text{ shop rivets, and}$$

$$\frac{2 \times 12\,300}{6\,013 \times 1\frac{1}{4}} = 4 \text{ field rivets.}$$

91. **The Lateral Systems.** The stresses in these systems must be computed according to (10) and Article 54, Part I. They are given on the stress sheet, Plate III (p. 251). Since according to (68) these members must be constructed of rigid shapes, it is customary, in computing the stresses, to assume that one-half the shear is taken by each of the diagonals in any given panel; that is, one diagonal is in tension, and the other diagonal is in compression. The stresses given on the stress sheet are computed by making this assumption. Also, since both diagonals in each panel are considered as acting at the same time, the stresses in all the verticals are zero.

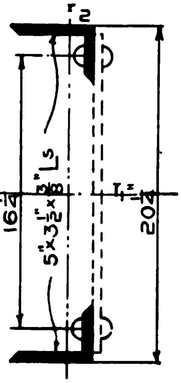


Fig. 193. Section of Upper Lateral Member.

The section of the upper lateral members will be made up of two angles placed apart a distance equal to the depth of the top chord. Fig. 193 shows the section. The radius of gyration about the axis parallel to the long leg will be considerably larger than that about an axis parallel to the shorter leg. In fact, it is so much greater that the strut will not need to be examined with respect to this axis. The diagram of the first panel is given in Fig. 194. The radius of gyration is to be taken about the horizontal axis if the entire length is to be taken; and the radius of gyration is to be taken about the vertical axis if one-half the length is taken, in which case it will bend as shown by the broken line in Fig. 194. The members are designed for the latter conditions only, since they are amply safe in regard to the first condition if they satisfy the latter. The length in this latter condition is 13.5 feet, which requires a radius of gyration not less than $\frac{13.5 \times 12}{120} = 1.35$.

Two angles 5 by 3 by ⅜-inch, with a total area of 5.72 square [illegible] 1.61, will be assumed and [illegible]

[illegible] $= [illegible] - 70 \times \left(\frac{13.5 \times 12}{1.61}\right) \times$ [illegible] and the required area is $\frac{6\,700}{11\,000} =$ [illegible] square inches. The required area is very much less than the [illegible] must be used, since this is the [illegible] of the Specifications.

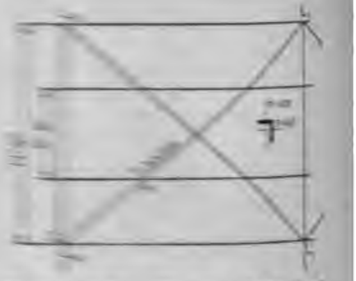


Fig. [illegible] Outline Diagram of [illegible]

Fig. [illegible] Outline Diagram of First Panel [illegible] System

[illegible] must be [illegible] inches; and according [illegible] the distance being [illegible] required. In accord- [illegible] = [illegible] Field [illegible] system is [illegible] will give a radius of [illegible] the remainder of the [illegible] minimum size might [illegible] that the [illegible]

[illegible] computed according [illegible] system being [illegible] stressed at the [illegible] compression. [illegible] system. These diagonals are connected to the stringers wherever they cross them, and also to each other where they cross in the center. This reduces the length which must be used in computing the cross-section of the member. In this case it is the distance C–A, and is equal to 90 inches.

Since the angle is free to move about either axis, angles with even legs should preferably be employed, since this will give greater economy. The radius of gyration must be greater than $\frac{90}{120} = 0.75$.

One angle $3\frac{1}{2}$ by $3\frac{1}{2}$ by $\frac{3}{8}$-inch, with an area of 2.48 square inches and a radius of gyration of 1.07, will be assumed and investigated. The allowable unit-stress is $P = (16\,000 - 70 \times \frac{90}{1.07})\ 1\frac{1}{4} = 12\,650$ pounds per square inch, and the required area is $\frac{30\,500}{12\,650} = 2.38$ square inches. This is nearly equal to the given area, and therefore the angle chosen will be taken for the section.

This angle must now be investigated for tension, one rivet-hole being taken out of the section. The net area is $2.48 - (\frac{7}{8} + \frac{1}{8})\ \frac{3}{8} = 2.10$ square inches. The required net area is $\frac{30\,500}{16\,000 \times 1\frac{1}{4}} = 1.53$ square inches, which shows the angle to be sufficiently strong.

Single shear determines the number of rivets to be required. These are:

$$\frac{2 \times 30\,500}{6\,013 \times 1\frac{1}{4}} = 8 \text{ field rivets.}$$

All rivets in the lower lateral system are field rivets, since this system also must be riveted up in the field after the trusses are swung into place.

The total stress in the second panel is 21 500 pounds, and a $3\frac{1}{2}$ by 3 by $\frac{3}{8}$-inch angle, with an area of 2.30 square inches and a least radius of gyration of 0.90, will be assumed and examined. The allowable unit-stress in compression is $1\frac{1}{4}\ (16\,000 - 70 \times \frac{90}{0.90}) = 11\,250$ pounds per square inch, and the required area is $\frac{21\,500}{11\,250} = 1.91$ square inches. Since this is less than the given area, and since the size of the angle (72) is the smallest allowable, this angle must be used.

It is required that this member shall have a net area of $\frac{21\,500}{16\,000 \times 1\frac{1}{4}} = 1.08$ square inches in tension. The net area of the angle, one rivet-hole being taken out, is 1.92 square inches, which shows the angle to be safe in tension.

The number of rivets required is determined by single shear, since they tend to shear off between the member itself and the connecting plates. The number required is $\frac{2 \times 21\,500}{6\,013 \times 1\frac{1}{4}} = 6$ field rivets.

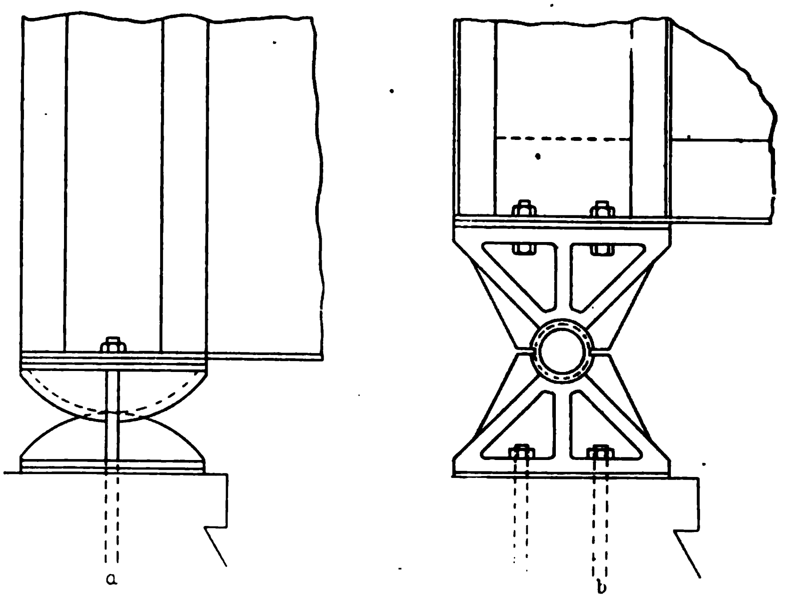


Fig. 196. Two Types of Bearings.

Since the above angle is the smallest that can be used, and since the remaining angles of the panel of the lateral bracing have smaller stresses than the one just designed, it is evident that this size angle must be used in all panels of the lower lateral system other than the first.

92. **The Shoes and Roller Nests.** For bridges of short spans and for plate-girders whose spans require rocker bearings to be provided (80), several different classes of bearings are in use. Two such bearings are shown in Fig. 196 (*a* and *b*). The type illustrated by *a* is seldom used on any spans except plate-girders. That shown in *b*

may be used on either plate-girders or small truss spans; it is the invention of Mr. F. E. Schall, Bridge Engineer of the Lehigh Valley Railroad, who uses it on plate-girders. It has given very great satisfaction; and for simplicity of design and also for economy it is to be recommended. Some railroads have used a bearing which consisted of a lens-shaped disc of phosphor-bronze, the faces of which fitted into corresponding indentations in both the masonry and the bearing plates. One advantage of this bearing is that it allows movements due to the deflection of the girder, and also lateral deflection of the floor-beam. It is claimed to have given satisfaction.

A bearing which is used on both short-span and long-span bridges is shown in Fig. 197. This class of bearing will be used. The end reaction of the bridge proper is equal to the vertical component of the stress in the end-post, and is $\frac{30.1}{36.7} \times 410\,500 = 336\,500$ pounds, which requires a bearing area (19) on the masonry, of $\frac{336\,500}{600} = 561$ square inches. According to the table on page 193, the masonry plate will be 28 inches long.

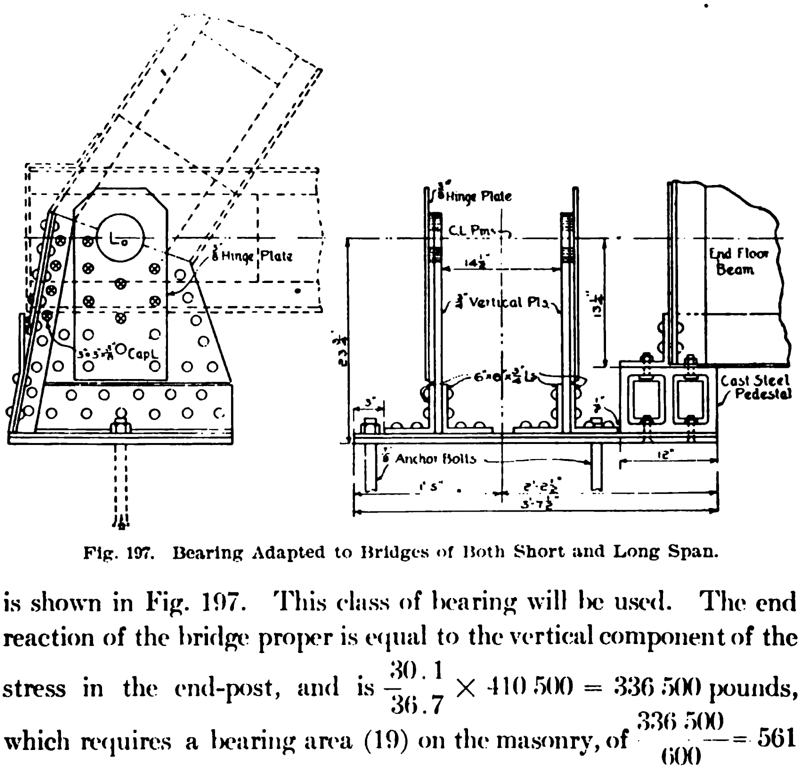


Fig. 197. Bearing Adapted to Bridges of Both Short and Long Span.

The total bearing area for one of the vertical plates is:

$$\frac{336\,500}{2 \times 24\,000} = 7.0 \text{ square inches;}$$

and the total required thickness is:

$$\frac{7.0}{6.25} = 1.12 \text{ inches,}$$

a 6¼-inch pin being used at L_0. Since the vertical plates will be made ¾ inch thick, this leaves a remainder of ⅜ inch to be made up of pin-plates.

The amount of stress which is carried by the ⅜-inch pin-plate is $\frac{0.375}{1.125} \times \frac{336\,500}{2} = 56\,100$ pounds. These plates will tend to shear off the rivets at a plane between the plates, and therefore $\frac{56\,100}{7\,220} = 8$ shop rivets will be required to fasten them to the vertical plate.

Since the length of the masonry plate is 28 inches, and the total area required is 561 square inches, the required width is $\frac{561}{28} = 20$ inches. The actual width will be greater than this, since it must be sufficient to allow for the connecting angles and also for the bearings of the end floor-beam. The connecting angles should be ¾ inch thick, and should not be less than 6 by 6 inches; and the plates to which they are connected should not be less than ¾ inch in thickness, and likewise they should not be greater, on account of the punching. The bottom plate should extend outward about 3 inches, in order to allow sufficient room for the anchor bolts, which should be ⅞ inch in diameter and should extend into the masonry at least 8 inches.

In addition to the reaction of the bridge proper, the masonry plate must be of sufficient area to give bearing for the end reaction of the end floor-beam. The maximum end reaction (see Article 83, p. 197) is 104 740 pounds. The bearing area required on the masonry is $\frac{104\,740}{600} = 175$ square inches; and assuming that the base of the bearing will be 12 inches long (see Fig. 197), the required length will be 14.6 inches. Usually, however, the bearing is extended the entire length of the masonry plate, which is 28 inches in this case.

The distance from the center of the pin to the top of the masonry will be the same for both the fixed and the roller end. This distance should be such that the angles of the shoe will clear the bottom chord member and allow the floor-beam to rest upon the plate as shown. Since the first section of the bottom chord is 18½ inches deep, the top of the angles of the two must be at least 9¼ inches from the center line of pins. This requires that the distance from the center line of the pin to the base of the angle shall be at least (9¼ + 6) = 15¼ inches, or more.

The tops of all floor-beams are at the same height, and the bottoms of the intermediate floor-beams must be on a level with the bottom of the first section of the lower chord (see Fig. 174). This requires that the bottom of the intermediate floor-beams shall be 9¼ inches below the center line of pins, and this brings the top of the floor-beams (48¼ − 9¼) = 39 inches above the center line of the pins. Since the end floor-beam is 52¼ inches deep, back to back of angles, the lower flange will be (52¼ − 39) = 13¼ inches below the center line of pins. In case the end floor-beam does not rest directly upon the bearing plate or the masonry plate, the intervening space is filled out with a grillage of iron bars or a cast-steel pedestal, as shown in Figs. 197 and 198.

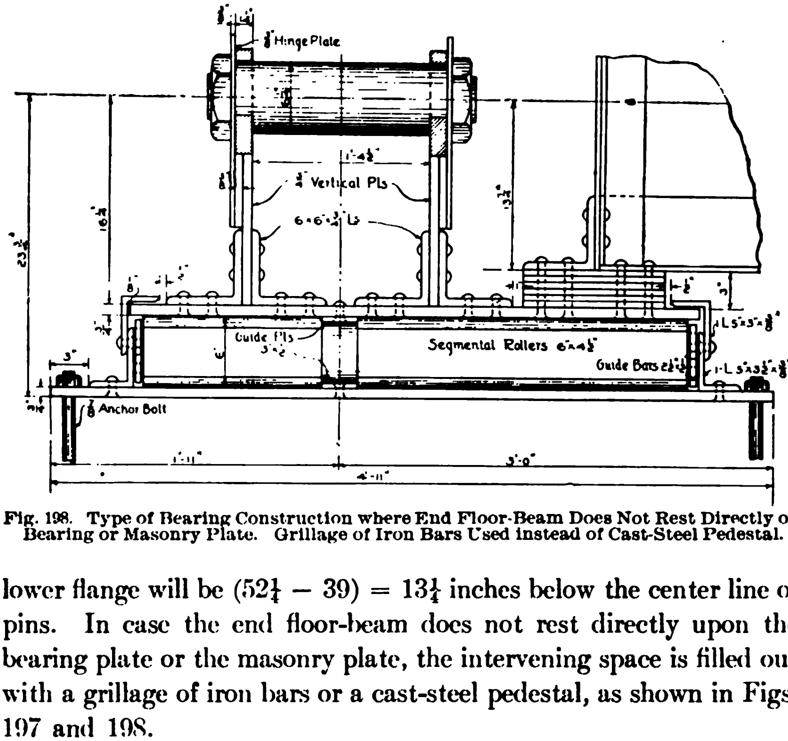


Fig. 198. Type of Bearing Construction where End Floor-Beam Does Not Rest Directly on Bearing or Masonry Plate. Grillage of Iron Bars Used instead of Cast-Steel Pedestal.

The small plates upon the side of the shoe, going entirely around the pin, are called the *shoe hinge-plates.* These do not take any stress, and require only sufficient rivets to hold them in position. They are used during erection to keep the end-post in line; and after erection their function is to keep the end-post on the shoe, and to prevent it from having any upward motion due to the vibration of the structure.

The rivets through the vertical legs of the shoe angles are in double bearing in the ¾-inch angles, in single bearing in the vertical plate, and in double shear. A rivet in double shear has a less value than in bearing in the plates. This value is 14 440 pounds, and therefore the number of shop rivets required through the vertical legs of the angles is:

$$\frac{336\,500}{2 \times 14\,440} = 12 \text{ rivets.}$$

The rivets which go through the horizontal leg of the angle and through the cap plate and cap angles, do not take stress. The number of rivets put in is that demanded by the detailing, the rivets in the horizontal legs of the angles usually staggering with those in the vertical legs. The cap plate tends to keep the vertical plates in line, and to keep out the dust and dirt and other deteriorating influences of the elements.

Wherever the rivet-heads tend to interfere with other members or project beyond surfaces which are required to be flat—as, for example, the bottom of the masonry or bearing plates—they must be countersunk (see Carnegie Handbook, p. 191, and "Steel Construction," Part III, p. 192).

The space for the anchor bolts, that for the connection angles, and that for the bearing of the end floor-beam, require that the total width of the masonry plate for the fixed end shall be $2 \times \frac{3}{4} + 14\frac{1}{2} + 2 \times 6 + \frac{1}{2} + 3 + 12 = 3$ feet $7\frac{1}{2}$ inches.

The design of the roller end requires that the length of the masonry bearing, the size of the vertical plates and angles, and also the number of rivets shall be the same as that for the fixed end. The width of the masonry plate is determined by the length of the rollers and their connections at the end.

The rollers (60) are required to be 6 inches in diameter, and the unit-stress (19) per linear inch is 6 × 600 = 3 600 pounds, which requires:

$$\frac{336\,500}{3\,600} = 93.5 \text{ linear inches.}$$

This is for the reaction of the bridge alone; and in addition to this, there are required for the floor-beam reaction:

$$\frac{104\,740}{3\,600} = 29.0 \text{ linear inches.}$$

The total number of linear inches is $93.5 + 29.0 = 122.5$; and if 5 rollers are used, they must be at least $\frac{122.5}{5} = 24.5$ inches long. The masonry plate is only 28 inches long, and therefore cylindrical rollers cannot be used, since they would occupy a space 30 inches or over. Segmental rollers (see Fig. 199) must be used.

The determination of the sizes of the angles which go at the end of the rollers, and also of the guide-plates, is a matter of judgment and experience. Those sizes indicated in Fig. 198, represent good engineering practice, and will be used.

The distance from the center line of pins to the top of the masonry can now be determined, and is $16\frac{1}{4} + \frac{3}{4} + 6 + \frac{3}{4} = 23\frac{3}{4}$ inches.

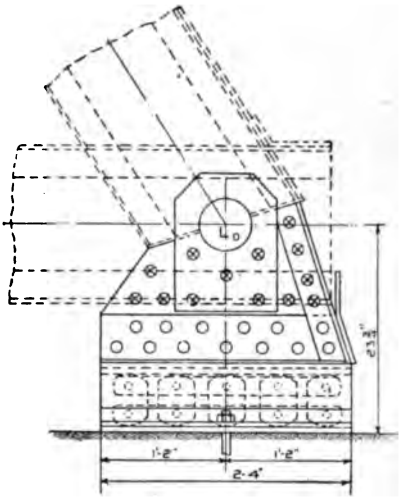


Fig. 199. Segmental Rollers Used for Bearings in Space under 30 Inches.

On account of putting in sufficient connections and angles as shown in Fig. 198, the masonry plate must be considerably wider than that theoretically determined. According to Fig. 198, the total width must be as follows, and the width should be computed in two parts, as the plate is not symmetrical about the center line of the truss:

From center line to outer edge:

$$\frac{14\frac{1}{2}}{2} + \frac{3}{4} + 6 + \frac{1}{2} + (3 - \tfrac{3}{8} = 2\tfrac{5}{8}) + (3\tfrac{1}{2} - \tfrac{3}{8} = 3\tfrac{1}{8}) + 3$$
$$= 1 \text{ ft. } 11\tfrac{1}{4} \text{ in., (say, 1 ft. 11 in.).}$$

From center line to inner edge:

$$\frac{14\frac{1}{2}}{2} + \frac{3}{4} + 6 + 1 + 12 + \frac{1}{2} + 2\tfrac{5}{8} + 3\tfrac{1}{8} + 3 = 3 \text{ ft. } 0\tfrac{1}{4} \text{ in., say, } 3 \text{ ft. } 0 \text{ in.}$$

Total width 4 ft. 11 in.

Allowing guide-plates and guide-bars of dimensions as shown in Fig. 198, and assuming $\frac{1}{8}$ inch as clearance at the ends, the total length of the rollers is:

$$(4 \text{ ft. } 11 \text{ in.}) - 2(3 + 3\tfrac{1}{2} + \tfrac{1}{8} + \tfrac{1}{8} + \tfrac{1}{2}) = 44.5 \text{ inches.}$$

This shows them to be amply long enough, as only 22 inches is [illegible] [illegible]. Here, as in most cases for single-track spans [illegible] 100 feet in length, the width of the masonry plate is determined [illegible] by the unit bearing stress.

The guide-plates are small bars riveted to the top of the bottom [illegible] and serve to keep the rollers in line. The guide-bars are connected to the rollers at their ends, and serve to keep the rollers equidistant, therefore causing them to roll easier and keeping them from coming into contact with each other.

The expansion [illegible] must be allowed for at the rate of ⅛ inch for every 10 feet in length of span. This makes a total allowed for temperature of expansion of $\frac{147}{10} \times \frac{1}{8} = 1\frac{7}{8}$, say 2 inches. No slotted holes are to be provided for the anchor bolts, since they do not go

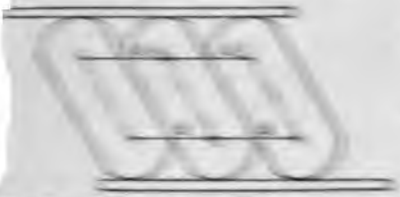


Fig. 2[illegible]. Bending of Insufficiently Spaced Segmental Rollers.

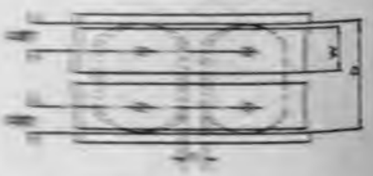


Fig. 2[illegible]. Determination of Spacing for Segmental Rollers.

[illegible] The shoe slides over [illegible] which are [illegible]

[illegible] the segmental rollers, [illegible] the extreme position [illegible] Fig. 2[illegible]. This distance can be [illegible] in Fig. 2[illegible] and from the following formula*:

$$[illegible]$$

$$[illegible]$$

[illegible] change of temperature, and D [illegible] taken in inches. The [illegible] is 2 inches; D is 6 inches;

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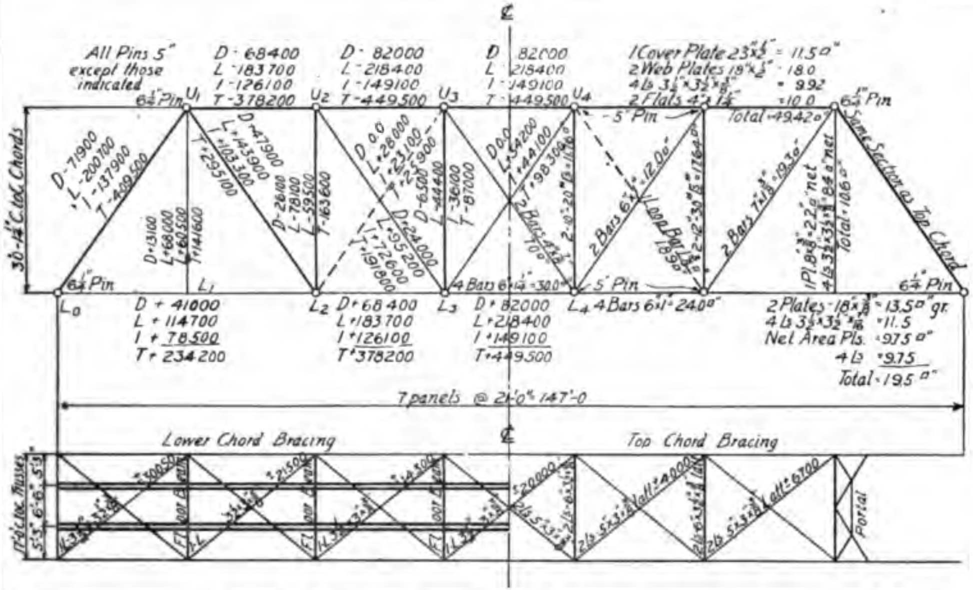
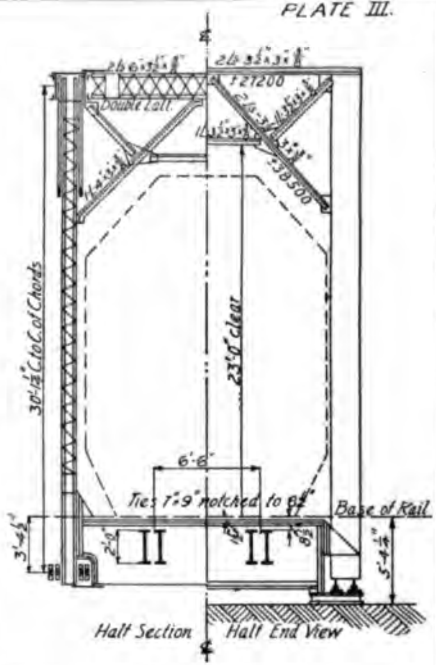
* [illegible] Prof. Frank [illegible] of Lehigh University, and published in *Engineering* [illegible]

PLATE III.





Stringers - 2 per rail 17'-0" C. to C. trusses 21'-0" panels

DLBM = 238000"# DL Shear = 3780#
LLBM = 2712000 LL Shear = 51400
Impact = 2510000 Impact = 48400
Total = 5460000"# Total = 103580#

Section Modulus = 5460000 ÷ 16000 = 341.0
Use 2-24"-80# Is - Section Mod. = 348.0

Floor Beams - 17'-0" C. to C. trusses

DLBM = 557200"# DL Shear = 9100#
LLBM = 4284000 LL Shear = 68000
Impact = 3814500 Impact = 60500
Total = 8655700"# Total = 137600#

4'-$0\frac{1}{4}$" b. to b. of Ls - 6"×6"×$\frac{5}{8}$" = 12.97 □" net
8655700 ÷ (44.57 × 16000) = 12.1 □" req. in flange

Stringer Connections

Use Ls - 6"×$3\frac{1}{2}$"×$\frac{3}{8}$"
12 field rivets required to connect to floor beam
7 shop rivets to webs of Is

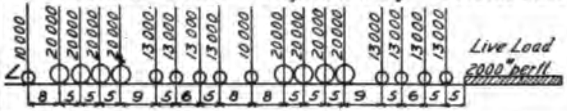
Floor Beam Connections

Use 6"×$3\frac{1}{2}$"×$\frac{3}{8}$" Ls
24 field rivets required to connect to posts
18 shop rivets to webs of floor beams

Roller End - Fl. Bm. Reaction = 104740#
End Reaction DL = 59000
LL = 164600
Use $\frac{7}{8}$" anchor bolts Imp. = 113000
Total = 441240#
441240 ÷ 600 = 736 □" req. bearing
441240 ÷ 3600 = 122.5" length of rollers
Plate 28"×59" = 1652 □" 5-6" eq. rollers 41" long

End Floor Beam b. to b. of Ls = 4'-$6\frac{1}{4}$"
DLBM = 318500"# DL Shear = 5340#
LLBM = 3238200 LL Shear = 51400
Impact = 3024000 Impact = 48000
Total = 6580700"# Total = 104740

4'-$6\frac{1}{4}$" b. to b. of Ls - 6"×6"×$\frac{7}{16}$" - 9.24 □" net
6579780 ÷ (48.93 × 16000) = 8.42 □" req. in flange



Specifications - American Railway Engineering and Maintenance of Way Assn. - 1906
Material M. O. H. Steel
Dead Load 1470 per lin. ft. of track

N. P. Ry No. 570 A
Single Track Thru. Pin Span
Over Mississippi R. at St. Paul, Minn.
147'-0" c. to c. end pins

Capital Bridge Co.
University Plant.

Cont. No.______ In Charge of______
Made by______ Date______
Checked by______ Date______
Order No.______ Sheet______

and ϕ, computed from the above formula, is 9° 30′. Substituting in the equation giving the value for y, there is obtained 1.02 inches (say 1¼ inches) for the distance between rollers. Rollers must not be less in thickness than the total expansion allowed for temperature.

Since there are 5 rollers, there are 4 spaces between them. Also, since the rollers must occupy a space of 28 inches, the length of the masonry plate, each roller must be:

$$\frac{28 - 4 \times 1\frac{1}{4}}{5} = 4.6 \text{ inches (say } 4\tfrac{1}{2} \text{ inches) wide.}$$

The width of the guide-bars must be such as to allow freedom of motion for the rollers. The maximum width allowable is given by the formula:*

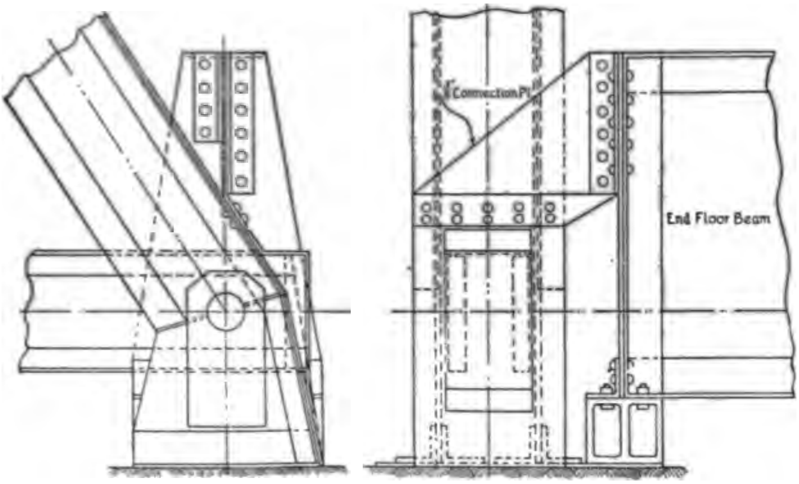


Fig. 202. Details of End Floor-Beam Connections.

$$W = \frac{D}{2} \cos \phi,$$

in which ϕ and D are indicated above. This requires the bar to be.

$$W = \frac{6}{2} \times 0.985 = 2.96 \text{ (say } 2\tfrac{3}{4}) \text{ inches wide.}$$

93. **The Stress Sheet.** Plate III shows the stress sheet of the bridge which has been designed in the preceding articles. This sheet represents the best current practice among the larger bridge

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* Derived by Prof. Frank B. McKibben of Lehigh University, and published in *Engineering News*, December, 1896.

corporations. It will be noted that very few details are given upon the sheet; also that few rivets are noted, and that sketches showing the manner in which the parts go together are entirely wanting. The shears and moments for the stringers and floor-beams, as well as the reactions and the number of rollers required, are given. This is to save the draftsman the trouble of recomputing values which have necessarily been determined by the designer.

The details of the various members, and also the manner in which the different members are connected, are left to the draftsman, who is under the direct supervision of the engineer in charge of the drafting room, upon whom rests the responsibility for good details. The figures given in the text indicate the best current practice. Figs.

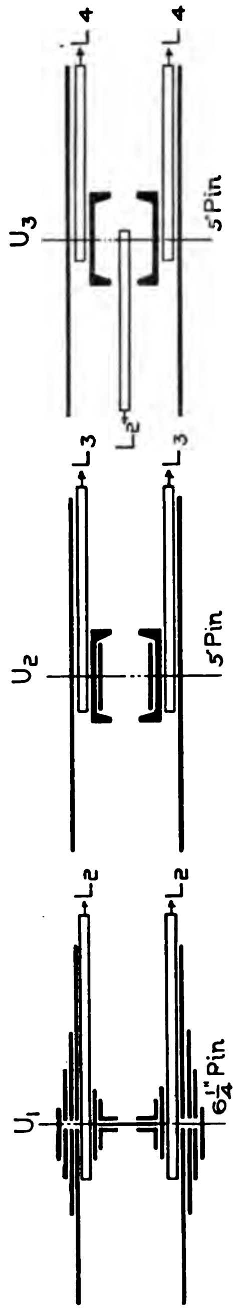


Fig. 203. Details of Packing of Upper Chord Members.

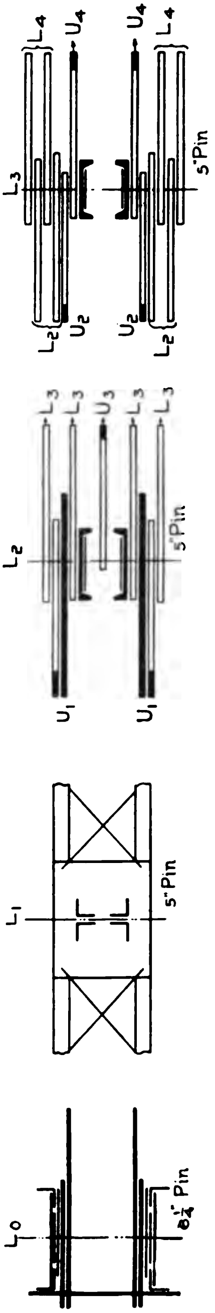
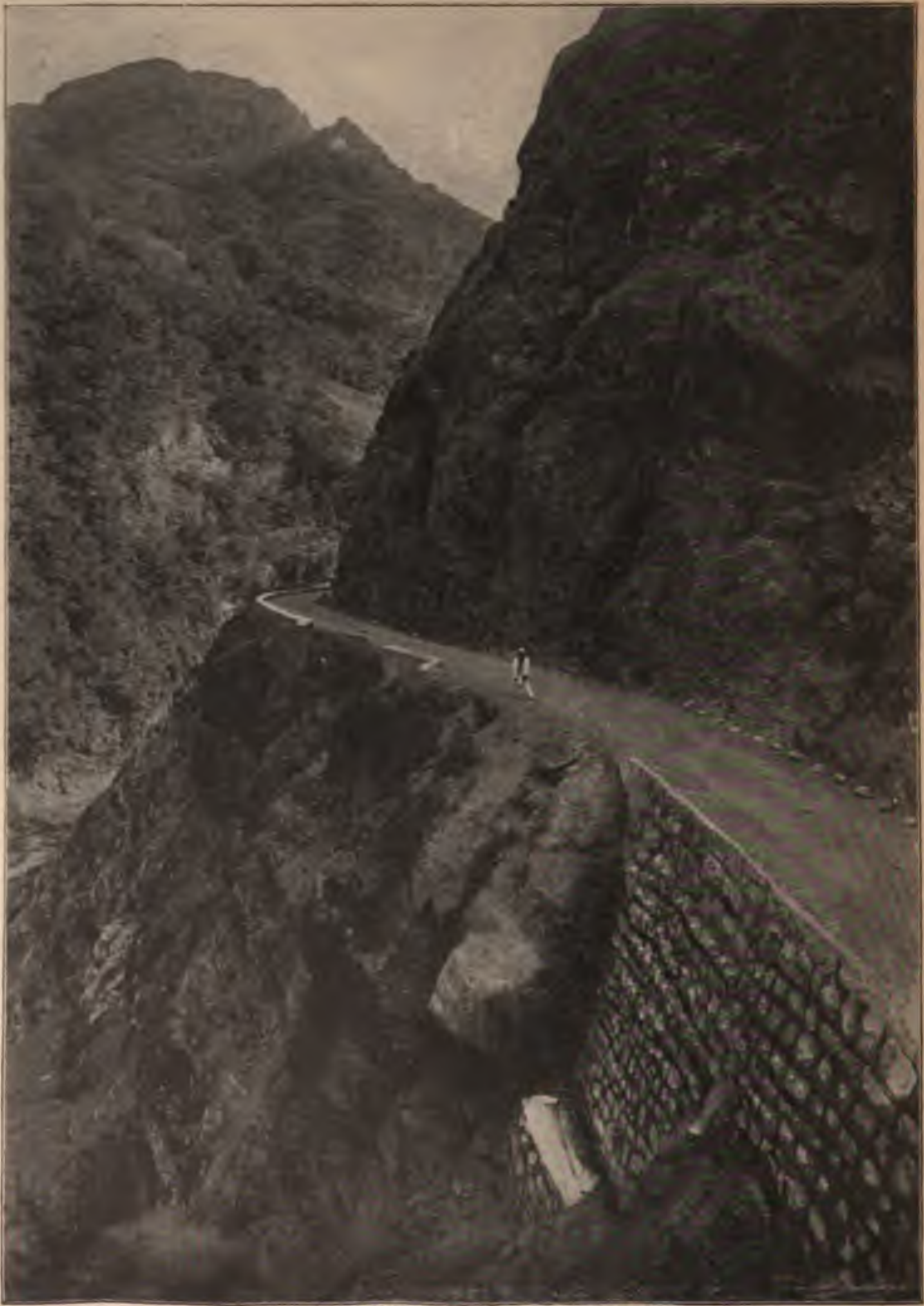


Fig. 204. Details of Packing of Lower Chord Members.

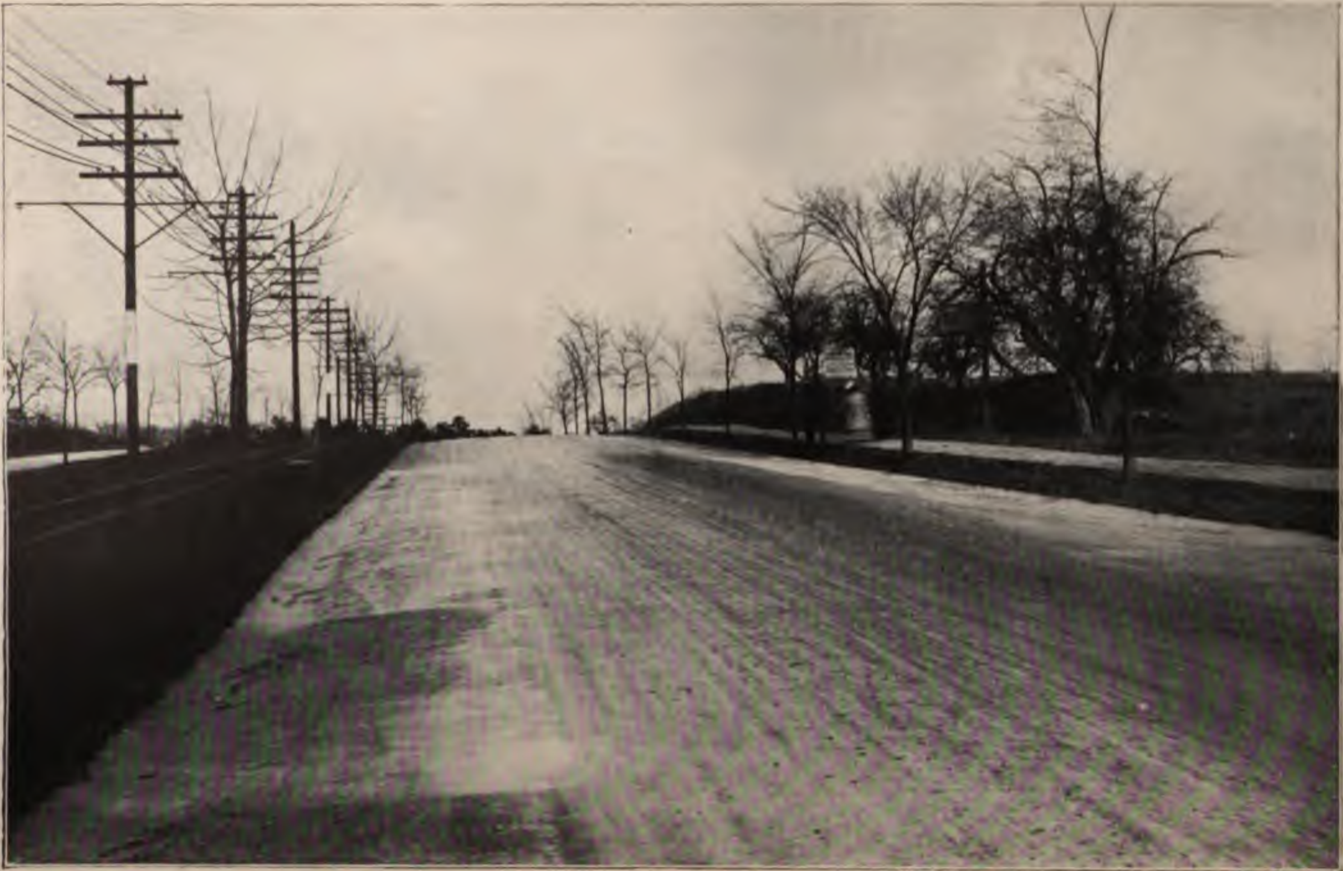


ROAD CONSTRUCTION IN THE PHILIPPINES

Section of 35-mile road built by American engineers, connecting the seaport of Dagupan with the mountain village of Baguio, province of Benguet, island of Luzón, and affording a cool and healthful retreat from the heat and malaria of the lowland regions. Dagupan lies 120 miles north of Manila, with which it is connected by rail. This view reveals some of the engineering difficulties to be overcome, masonry and concrete work of the type shown being necessary at many points.

THE NEW
PUBLIC LIB

ASTOR
TILDEN



A PORTION OF THE NEWTON BOULEVARD, NEWTON, MASSACHUSETTS

Opposite the Brae Burn Club. A fine example of macadamized road, treated with "Tarvia," an anti-dust compound.

HIGHWAY CONSTRUCTION

PART I.

COUNTRY ROADS.

GENERAL CONSIDERATIONS.

Object of Roads. The object of a road is to provide a way for the transportation of persons and goods from one place to another with the least expenditure of power and expense. The facility with which this traffic or transportation may be conducted over any given road depends upon the resistance offered to the movement of vehicles. This resistance is composed of: (1) The resistance offered by the roadway, which consists of (*a*) "friction" between the surface of the road and the wheel tires; (*b*) resistance offered to the rolling of the wheels, occasioned by the want of uniformity in the road surface, or lack of strength to resist the penetrating effects of the loaded wheels, thus requiring the load to be lifted over projecting points and out of hollows and ruts, thereby diminishing the effective load the horse may draw to such as it can lift. This resistance is called "resistance to rolling" or "penetration;" (*c*) resistance due to gravity, called "grade resistance;" (2) The resistance offered by vehicles, termed "axle friction;" (3) Resistance of the air.

The road which offers the least resistance to traffic should combine a surface on which the friction of the wheels is reduced to the least possible amount, while offering a good foothold for horses, to enable them to exert their utmost tractive power, and should be so located as to give the most direct route with the least gradients.

Friction. The resistance of friction arises from the rubbing of the wheel tires against the surface of the road. This resistance to traction is variable, and can be determined only by experiment. From many experiments the following deductions are drawn:

(1) The resistance to traction is directly proportional to the pressure

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(2) On solid, unyielding surfaces it is independent of the width of the tire, but on compressible surfaces the resistance decreases as the width of the tire increases (but there is no material advantage gained in making a tire more than 4 inches wide).

(3) It is independent of the speed.

(4) On rough, irregular surfaces, which give rise to constant concussion, it increases with the speed.

The following table shows the relative resistance to traction of various surfaces:

TABLE 1.

Resistance to Traction on Different Road Surfaces.

	Traction Resistance.	
	Pounds per ton.	In terms of load.
Earth road—ordinary condition	50 to 200	$\frac{1}{40}$ to $\frac{1}{10}$
Gravel	50 to 100	$\frac{1}{40}$ to $\frac{1}{20}$
Sand	100 to 200	$\frac{1}{20}$ to $\frac{1}{10}$
Macadam	30 to 100	$\frac{1}{67}$ to $\frac{1}{20}$
Plank Road	30 to 50	$\frac{1}{67}$ to $\frac{1}{40}$
Steel Wheelway	15 to 40	$\frac{1}{133}$ to $\frac{1}{50}$

These coefficients refer to the power required to keep the load in motion. It requires from two to six or eight times as much force to start a load as it does to keep it in motion, at two or three miles per hour. The extra force required to start a load is due in part to the fact that during the stop the wheel may settle into the road surface, in part to the fact that the axle friction at starting is greater than after motion has begun, and further in part to the fact that energy is consumed in accelerating the load.

Resistance to Rolling. This resistance is caused (1) by the wheel penetrating or sinking below the surface of the road, leaving a track or rut behind it. It is equal to the product of the load multiplied by one-third of the semi-chord of the submerged arc of the wheel; and (2) by the wheel striking or colliding with loose or projecting stones, which give a sudden check to the horses, depending upon the height of the obstacle, the momentum destroyed being oftentimes considerable.

The rolling resistance varies inversely as some **function of the** diameter of the wheel, as the larger the wheel the less force required to lift it over the obstruction or to roll it up the inclination due to the indentation of the surface.

The power required to draw a wheel over a stone or any obstacle, such as S in Fig. 1, may be thus calculated. Let P represent the power sought, or that which would just balance the weight on the point of the stone, and the slightest increase of which would draw it over. This power acts in the direction C P with the leverage of B C or D E. Gravity, represented by W, resists in the direction C B with the leverage B D. The equation of equilibrium will be P × C B = W × B D, whence

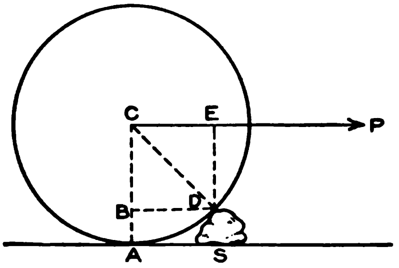


Fig. 1.

$$P = W\frac{BD}{CB} = W\frac{\sqrt{CD^2 - BC^2}}{CD - AD}$$

Let the radius of the wheel = C D = 26 inches, and the height of the obstacle = A B = 4 inches. Let the weight W = 500 pounds, of which 200 pounds may be the weight of the wheel and 300 pounds the load on the axle. The formula then becomes

$$P = 500\frac{\sqrt{676 - 484}}{26 - 4} = 500\frac{13.85}{22} = 314.7 \text{ pounds.}$$

The pressure at the point D is compounded of the weight and the power, and equals

$$W\frac{CD}{CB} = 500 \times \frac{26}{22} = 591 \text{ pounds,}$$

and therefore acts with this great effect to destroy the road in its collision with the stone, in addition there is to be considered the effect of the blow given by the wheel in descending from it. For minute accuracy the non-horizontal direction of the draught and the thickness of the axle should be taken into account. The power required is lessened by proper springs to vehicles, by enlarged wheels, and by making the line of draught ascending.

The mechanical advantage of the wheel in surmounting an obstacle may be computed from the principle of the lever.

Let the wheel, Fig. 2, touch the horizontal line of traction in the point A and meet a protuberance B D. Suppose the line of draught C P to be parallel to A B. Join C D and draw the perpendiculars D E and D F. We may suppose the power to be applied at E and the weight at F, and the action is then the same as the bent lever E D F turning round the fulcrum at D. Hence P : W :: F D : D E. But F D : D E :: tan F C D : 1, and tan F C D = tan 2 (D A B); therefore P = W tan 2 (D A B). Now it is obvious that the angle D A B increases as the radius of the circle diminishes; and therefore, the weight W being constant, the power required to overcome an obstacle of given height is diminished when the diameter is increased. Large wheels are therefore the best adapted for surmounting inequalities of the road.

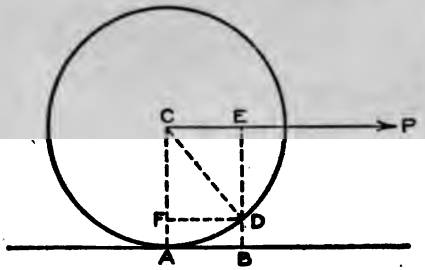


Fig. 2.

There are, however, circumstances which provide limits to the height of the wheels of vehicles. If the radius A C exceeds the height of that part of the horse to which the traces are attached, the line of traction C P will be inclined to the horse, and part of the power will be exerted in pressing the wheel against the ground. The best average size of wheels is considered to be about 6 feet in diameter.

Wheels of large diameter do less damage to a road than small ones, and cause less draught for the horses.

With the same load, a two-wheeled cart does far more damage than one with four wheels, and this because of their sudden and irregular twisting motion in the trackway.

Grade Resistance is due to the action of gravity, and is the same on good and bad roads. On level roads its effect is immaterial, as it acts in a direction perpendicular to the plane of the horizon, and neither accelerates nor retards motion. On inclined roads it offers considerable resistance, proportional to the steepness of the incline.

The resistance due to gravity on any incline in pounds per ton is equal to $\frac{2000}{\text{rate of grade}}$.

The following table shows the resistance due to gravity on different grades.

TABLE 2.

Resistance Due to Gravity on Different Inclinations.

Grade 1 in............	20	30	40	50	60	70	80	90	100	200	300	400
Rise in feet per mile ...	264	176	132	105	88	75	66	58	52	26	17	13
Resistance in lb. per ton .	112	74½	56	45	38	32	28	25	22	11¼	7½	5½

The additional resistance caused by inclines may be investigated in the following manner: Suppose the whole weight to be borne on one pair of wheels, and that the tractive force is applied in a direction parallel to the surface of the road.

Let A B in Fig. 3 represent a portion of the inclined road, C being a vehicle just sustained in its position by a force acting in the direction C D. It is evident that the vehicle is kept in its position by three forces; namely, by its own weight W acting in the vertical direction C F, by the force F applied in the direction C D parallel to the surface of the road, and by the pressure P which the vehicle exerts against the surface of the road acting in the direction C E perpendicular to same. To determine the relative magnitude of these three forces, draw a horizontal line A G and the vertical one B G; then, since the two lines C F and B G are parallel and are both cut by the line A B, they must make the two angles C F E and A B G equal; also the two angles C E F and A G B are equal; therefore, the remaining angles F C E and B A G are equal, and the two triangles C F E and A B G are similar. And as the three sides of the former are proportional to the three forces by which the vehicle is sustained, so also are the three sides of the latter; namely, A B or the length of the road is proportional to W, or the weight of the vehicle; B G,

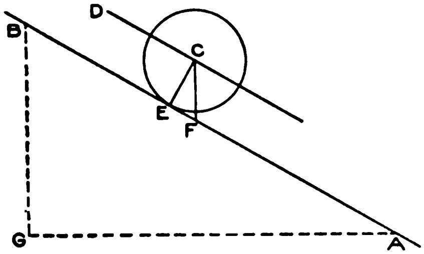


Fig. 3.

extent be vague, owing to the different strengths and speeds of animals of the same kind, as well as to the extent of their training to any particular kind of work.

The draught or pull which a good average horse, weighing 1,200 pounds, can exert on a level, smooth road at a speed of 2½ miles per hour is 100 pounds, equivalent to 22,000 foot-pounds per minute, or 13,200,000 foot-pounds per day of 10 hours.

The tractive power diminishes as the speed increases and, perhaps, within certain limits, say from ¾ to 4 miles per hour, nearly in inverse proportion to it. Thus the average tractive force of a horse, on a level, and actually pulling for 10 hours, may be assumed approximately as follows:

TABLE 3.
Tractive Power of Horses at Different Velocities.

Miles per hour.	Tractive Force. Lb.	Miles per hour.	Tractive Force. Lb.
¾	333.33	2¼	111.11
1	250	2½	100
1¼	200	2¾	90.91
1½	166.66	3	83.33
1¾	142.86	3½	71.43
2	125	4	62.50

The work done by a horse is greatest when the velocity with which he moves is ⅓ of the greatest velocity with which he can move when unloaded; and the force thus exerted is 0.45 of the utmost force that he can exert at a dead pull.

The traction power of a horse may be increased in about the same proportion as the time is diminished, so that when working from 5 to 10 hours, on a level, it will be about as shown in the following table:

TABLE 4.

Hours per day	Traction (pounds)	Hours per day	Traction (pounds)
10	100	7	146⁶⁄₇
9	111⅑	6	166⅓
8	125	5	200

The tractive power of teams is about as follows

1 horse .... = 1
2 horses .... 0.95 × 2 = 1.90
3 " .... 0.85 × 3 = 2.55
4 " .... 0.80 × 4 = 3.20

Loss of Tractive Power on Inclines. In ascending inclines a horse's power diminishes rapidly; a large portion of his strength is expended in overcoming the resistance of gravity due to his own weight and that of the load. Table 5 shows that as the steepness of the grade increases the efficiency of both the horse and the road surface diminishes; that the more of the horse's energy is expended in overcoming gravity the less remains to overcome the surface resistance.

TABLE 5.

Effects of Grades Upon the Load a Horse can Draw on Different Pavements.

Grade.	Earth.	Broken Stone.	Stone Blocks.	Asphalt.
Level	1.00	1.00	1.00	1.00
1 : 100	.80	.66	.72	.41
2 : 100	.66	.50	.55	.25
3 : 100	.55	.40	.44	.18
4 : 100	.47	.33	.36	.13
5 : 100	.41	.29	.30	.10
10 : 100	.26	.16	.14	.04
15 : 100	.10	.05	.07	...
20 : 100	.04	...	.03	...

Table 6 shows the gross load which an average horse, weighing 1,200 pounds, can draw on different kinds of road surfaces, on a level and on grades rising five and ten feet per one hundred feet.

TABLE 6.

Description of Surface	Level.	5 per cent grade.	10 per cent grade.
Asphalt	13,216	....	....
Broken stone (best condition)	6,700	1,840	1,060
" " (slightly muddy)	4,700	1,500	1,000
" " (ruts and mud)	3,000	1,390	890
" " (very bad condition)	1,840	1,040	740
Earth (best condition)	3,600	1,500	930
" (average condition)	1,400	900	660
" (moist but not muddy)	1,100	780	600
Stone-block pavement (dry and clean)	8,300	1,920	1,090
" " " (muddy)	6,250	1,800	1,040
Sand (wet)	1,500	675	390
" (dry)	1,087	445	217

The decrease in the load which a horse can draw upon an incline is not due alone to gravity; it varies with the amount of foothold afforded by the road surface. The tangent of the angle of inclination should not be greater than the coefficient of tractional resistance; therefore it is evident that the smoother the road surface, the easier should be the grade. The smoother the surface the less the foothold, and consequently the load.

The loss of tractive power on inclines is greater than any investigation will show; for, besides the increase of draught caused by gravity, the power of the horse is much diminished by fatigue upon a long ascent, and even in greater ratio than man, owing to its anatomical formation and great weight. Though a horse on a level is as strong as five men, on a grade of 15 per cent, it is less strong than three; for three men carrying each 100 pounds will ascend such a grade faster and with less fatigue than a horse with 300 pounds.

A horse can exert for a short time twice the average tractive pull which he can exert continuously throughout the day's work; hence, so long as the resistance on the incline is not more than double the resistance on the level, the horse will be able to take up the full load which he is capable of drawing.

Steep grades are thus seen to be objectionable, and particularly so when a single one occurs on an otherwise comparatively level road, in which case the load carried over the less inclined portions must be reduced to what can be hauled up the steeper portion.

The bad effects of steep grades are especially felt in winter, when ice covers the roads, for the slippery condition of the surface causes danger in descending, as well as increased labor in ascending; the water of rains also runs down the road and gulleys it out, destroying its surface, thus causing a constant expense for repairs. The inclined portions are subject to greater wear from horses ascending, thus requiring thicker covering than the more level portions, and hence increasing the cost of construction.

It will rarely be possible, except in a flat or comparatively level country, to combine easy grades with the best and most direct route. These two requirements will often conflict. In such a case, increase the length. The proportion of this increase will depend upon the friction of the covering adopted. But no general rule can be given to meet all cases as respects the length which may thus be added, for the comparative time occupied in making the journey forms an important element in any case which arises for settlement. Disregarding time, the horizontal length of a road may be increased to avoid a 5 per cent grade, seventy times the height.

Table 7 shows, for most practical purposes, the force required to draw loaded vehicles over inclined roads. The first column expresses the rate of inclination; the second, the pressure on the plane in pounds per ton; the third, the tendency down the plane (or force required to overcome the effect of gravity) in pounds per ton; the fourth, the force required to haul one ton up the incline; the fifth, the length of level road which would be equivalent to a mile in length of the inclined road—that is, the length which would require the same motive power to be expended in drawing the load over it, as would be necessary to draw over a mile of the inclined road; the sixth, the maximum load which an average horse weighing 1,200 pounds can draw over such inclines, the friction of the surface being taken at $\frac{1}{50}$ of the load drawn.

TABLE 7.

Rate of grade. Feet per 100 feet.	Pressure on the plane in lb. per ton.	Tendency down the plane in lb. per ton.	Power in lb. required to haul one ton up the plane.	Equivalent length of level road. Miles.	Maximum load in lb. which a horse can haul.
0.0	2240	.00	45.00	1.000	6270
0.25	2240	5.60	50.60	1.121	5376
0.50	2240	11.20	56.20	1.242	4973
0.75	2240	16.80	61.80	1.373	4490
1.	2240	22.40	67.40	1.500	4145
1.25	*2240	28.00	73.00	1.622	3830
1.50	2240	33.60	78.60	1.746	3584
1.75	2240	39.20	84.20	1.871	3290
2	2240	45.00	90.00	2.000	3114
2.25	2240	50.40	95.40	2.120	2935
2.50	2240	56.00	101.00	2.244	2725
2.75	2240	61.33	106.33	2.363	2620
3	2239	67.20	112.20	2.484	2486
4	2238	89.20	134.20	2.982	2083
5	2237	112.00	157.00	3.444	1800
6	2233	134.40	179.40	3.986	1568
7	2232	156.80	201.80	4.844	1367
8	2232	179.20	224.20	4.982	1235
9	2231	201.60	246.60	5.840	1125
10	2229	224.00	269.00	5.977	1030

* Near enough for practice; actually 2239.888.

Pressure on the plane = weight × nat cos of angle of plane.

Axle Friction. The resistance of the hub to turning on the axle is the same as that of a journal revolving in its bearing, and has nothing to do with the condition of the road surface. The coefficient of journal friction varies with the material of the journal and its bearing, and with the lubricant. It is nearly independent of the velocity, and seems to vary about inversely as the square root of the pressure. For light carriages when loaded, the coefficient of friction is about 0.020 of the weight on the axle; for the ordinary thimble-skein wagon when loaded, it is about 0.012. These coefficients are for good lubrication; if the lubrication is deficient, the axle friction is two to six times as much as above.

The traction power required to overcome the above axle friction for carriages of the usual proportions is about 3 to 3½ lb. per ton of the weight on the axle; and for truck wagons, which have medium sized wheels and axles, is about 3½ to 4½ lb. per ton.

Resistance of the Air. The resistance arising from the force of the wind will vary with the velocity of the wind, with the velocity of the vehicle, with the area of the surface acted upon, and also with the angle of incidence of direction of the wind with the plane of the surface.

The following table gives the force per square foot for various velocities:

TABLE 8.

Velocity of wind in miles per hour.	Force in lbs. per sq. ft.	Description.
15	1.107	Pleasant Breeze
20	1.968	Brisk Gale
25	3.075	
30	4.428	High Wind
35	6.027	
40	7.872	Very High Wind
45	9.963	
50	12.300	Storm

Effect of Springs on Vehicles. Experiments have shown that vehicles mounted on springs materially decrease the resistance to traction, and diminish the wear of the road, especially at speeds beyond a walking pace. Going at a trot, they were found not to cause more wear than vehicles without springs at a walk, all other conditions being similar. Vehicles with springs improperly fixed cause considerable concussion, which in turn destroys the road covering.

LOCATION OF COUNTRY ROADS.

The considerations governing the location of country roads are dependent upon the commercial condition of the country to be traversed. In old and long-inhabited sections the controlling elements will be the character of the traffic to be accommodated. In such a section, the route is generally predetermined, and therefore there is less liberty of a choice and selection than in a new and sparsely settled district, where the object is to establish the easiest, shortest, and most economical line of intercommunication according to the physical character of the ground.

Whichever of these two cases may have to be dealt with, the same principle governs the engineer, namely, to so lay out the road as to effect the conveyance of the traffic with the least expenditure of motive power consistent with economy of construction and maintenance.

Economy of motive power is promoted by easy grades, by the avoidance of all unnecessary ascents and descents, and by a direct line; but directness must be sacrificed to secure easy grades and to avoid expensive construction.

Reconnoissance. The selection of the best route demands much care and consideration on the part of the engineer. To obtain the requisite data upon which to form his judgment, he must make a personal reconnoissance of the district. This requires that the proposed route be either ridden or walked over and a careful examination made of the principal physical contours and natural features of the district. The amount of care demanded and the difficulties attending the operations will altogether depend upon the character of the country.

The immediate object of the reconnoissance is to select one or more trial lines, from which the final route may be ultimately determined.

When there are no maps of the section traversed, or when those which can be procured are indefinite or inaccurate, the work of reconnoitering will be much increased.

In making a reconnoissance there are several points which, if carefully attended to, will very considerably lessen the labor and time otherwise required. Lines which would run along the immediate bank of a large stream must of necessity intersect all the tributaries confluent on that bank, thereby demanding a corresponding number of bridges. Those, again, which are situated along the slopes of hills are more liable in rainy weather to suffer from washing away of the earthwork and sliding of the embankments; the others which are placed in valleys or elevated plateaux, when the line crosses the ridges dividing the principal water courses will have steep ascents and descents.

In making an examination of a tract of country, the first point to attract notice is the unevenness or undulations of its surface, which appears to be entirely without system, order, or arrangement; but upon closer examination it will be perceived that one general principle of configuration obtains even in the most irregular countries. The country is intersected in various directions by main water courses or rivers, which increase in size as they approach the point of their discharge. Towards these main rivers lesser rivers approach on both sides, running right and left through the country, and into these, again, enter still smaller streams and brooks. The streams thus divide the hills into branches or spurs having approximately the same direction as themselves, and the ground falls in every direction from the main chain of hills towards the water courses, forming ridges more or less elevated.

The main ridge is cut down at the heads of the streams into depressions called gaps or passes; the more elevated points are called peaks. The water which has fallen upon these peaks is the origin of the streams which have hollowed out the valleys. Furthermore, the ground falls in every direction towards the natural water courses, forming ridges more or less elevated running between them and separating from each other the districts drained by the streams.

The natural water courses mark not only the lowest lines, but the lines of the greatest longitudinal slope in the valleys through which they flow.

The direction and position of the principal streams give also the direction and approximate position of the high ground or ridges which lie between them.

The positions of the tributaries to the larger stream generally indicate the points of greatest depression in the summits of the ridges,

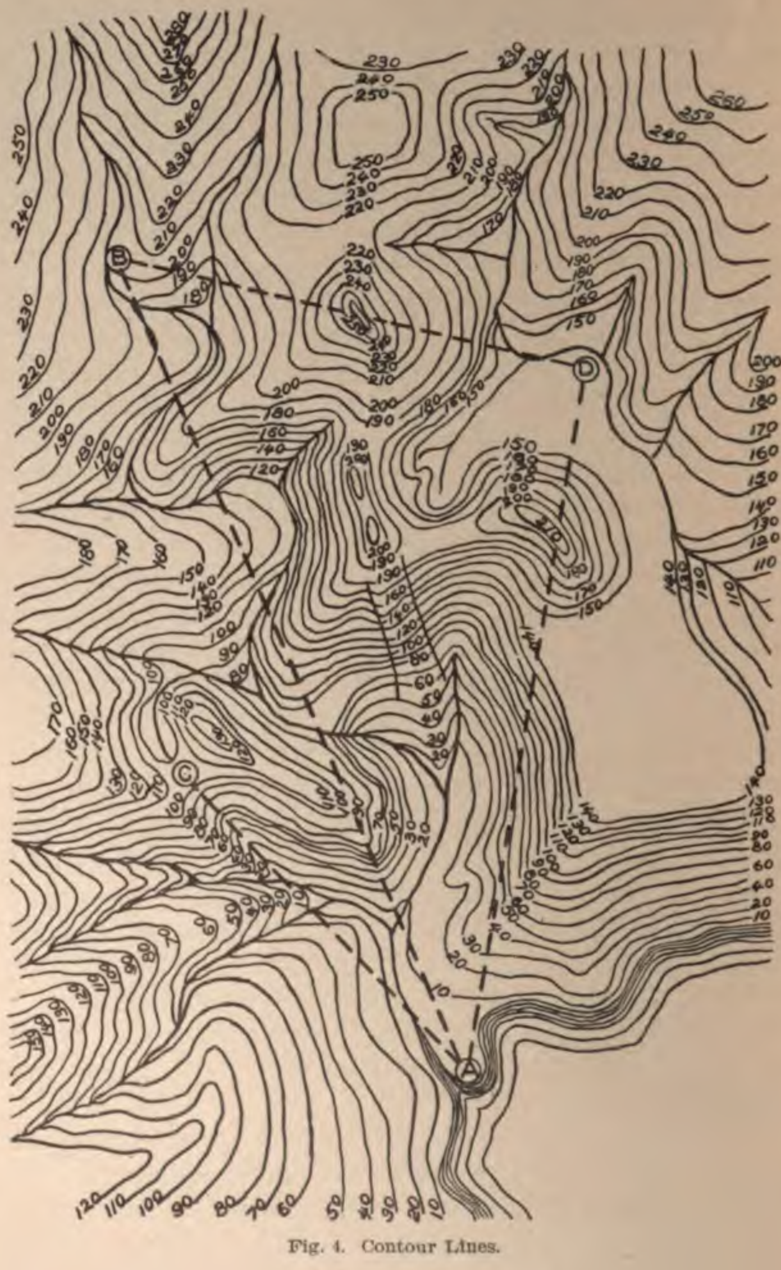


Fig. 4. Contour Lines.

and therefore the points at which lateral communication across the high ground separating contiguous valleys can be most readily made.

The instruments employed in reconnoitering, are: The compass, for ascertaining the direction; the aneroid barometer, to fix the approximate elevation of summits, etc.; and the hand level, to ascertain the elevation of neighboring points. If a vehicle can be used, an odometer may be added, but distances can usually be guessed or ascertained by time estimates or otherwise, closely enough for preliminary purposes. The best maps obtainable and traveling companions who possess a local knowledge of the country, together with the above outfit is all that will be necessary for the first inspection.

The reconnoissance being completed, instrumental surveys of the routes deemed most advantageous should be made. When the several lines are plotted to the same scale, a good map can be prepared from which the exact location of the road can be determined.

In making the preliminary surveys the topographical features should be noted for a convenient distance to the right and the left of the line, and all prominent points located by compass bearings. The following data should also be obtained: the importance, magnitude, and direction of all streams and roads crossed; the character of the material to be excavated or available for embankments, the position of quarries and gravel pits, and the modes of access thereto; and all other information that may effect a selection.

Topography. There are various methods of delineating upon paper the irregularities of the surface of the ground. The method of most utility to the engineer is that by means of "contour lines." These are fine lines traced through the points of equal level over the surface surveyed, and denote that the level of the ground throughout the whole of their course is identical; that is to say, that every part of the ground over which the line passes is at a certain height above a known fixed point termed the datum, this height being indicated by the figures written against the line.

The intervals between the lines vertically are equal and may be 1, 3, 5, 10 or more feet apart; where the surface is very steep they lie close together. These lines by their greater or less distance apart have the effect of shading, and make apparent to the eye, the undulations and irregularities in the surface of the country.

Fig. 4 shows an imaginary tract of country, the physical features of which are shown by contour lines.

Map. The map should show the lengths and direction of the different portions of the line, the topography, rivers, water courses, roads, railroads, and other matters of interest, such as town and county lines, dividing lines between property, timbered and cultivated lands, etc.

Any convenient scale may be adopted; 400 feet to an inch will be found the most useful.

Memoir. The descriptive memoir should give with minuteness all information, such as the nature of the soil, character of the several excavations whether earth or rock, and such particular features as cannot be clearly shown upon the map or profile.

Special information should be given regarding the rivers crossed, as to their width, depth at highest known flood, velocity of current, character of banks and bottom, and the angle of skew which the course makes with the line of the road.

Levels. Levels should be taken along the course of each line, usually at every 100 feet, or at closer intervals, depending upon the nature of the country.

In taking the levels, the heights of all existing roads, railroads, rivers, or canals should be noted. "Bench marks" should be established at least every half mile, that is, marks made on any fixed object, such as a gate post, side of a house, or, in the absence of these, a cut made on a large tree. The height and exact

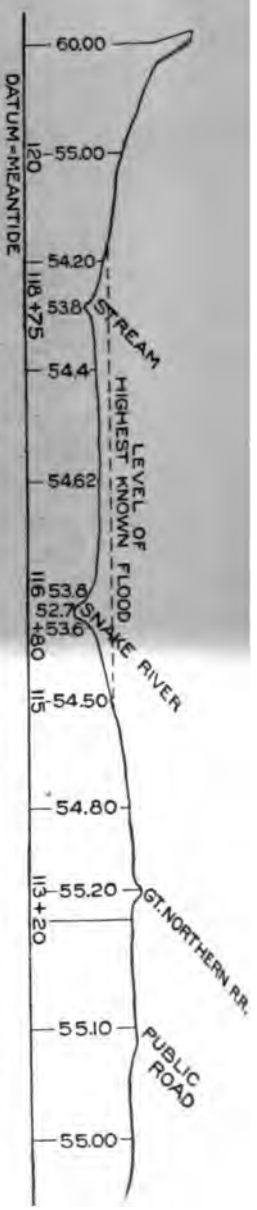
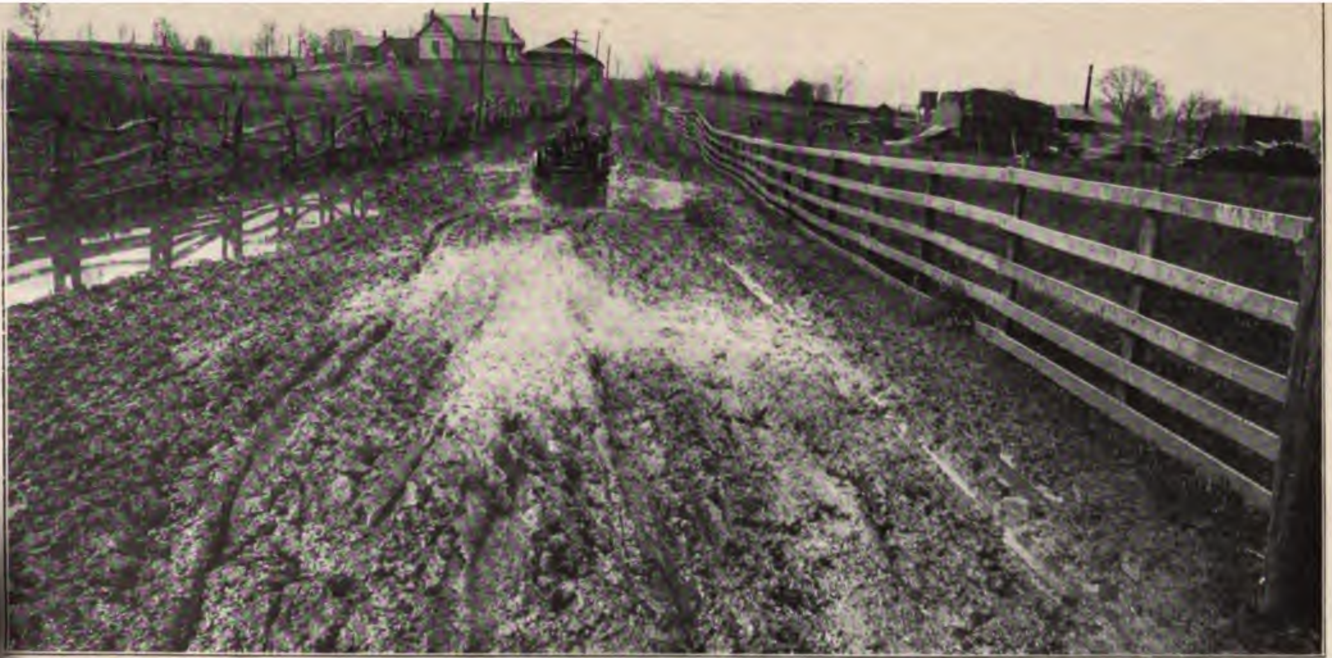


Fig. 5. Preliminary Profil



ROAD AT JOHNSON CITY, TENNESSEE, BEFORE MACADAMIZING

This almost impassable highway afforded an excellent opportunity to the Office of Public Roads of the U. S. Department of Agriculture to demonstrate the possibilities of road improvement by macadamizing with limestone. Various stages in the improvement that was carried out are illustrated elsewhere (see pages 286, 295, and 299).

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description of each bench mark should be recorded in the level book.

Cross Levels. Wherever considered necessary levels at right angle to the center line should be taken. These will be found useful in showing what effect a deviation to the right or left of the surveyed line would have. Cross levels should be taken at the intersection of all roads and railroads to show to what extent, if any, these levels will have to be altered to suit the levels of the proposed road.

Profile. A profile is a longitudinal section of the route, made from the levels. Its horizontal scale should be the same as that of the map; the vertical scale should be such as will show with distinctness the inequalities of the ground.

Fig. 5 shows the manner in which a profile is drawn and the nature of the information to be given upon it.

Bridge Sites. The question of choosing the site of bridges is an important one. If the selection is not restricted to a particular point, the river should be examined for a considerable distance above and below what would be the most convenient point for crossing; and if a better site is found, the line of the road must be made subordinate to it. If several practicable crossings exist, they must be carefully compared in order to select the one most advantageous. The following are controlling conditions: (1) Good character of the river bed, affording a firm foundation. If rock is present near the surface of the river bed, the foundation will be easy of execution and stability and economy will be insured. (2) Stability of river banks, thus securing a permanent concentration of the waters in the same bed. (3) The axis of the bridge should be at right angles to the direction of the current. (4) Bends in rivers are not suitable localities and should be avoided if possible. A straight reach above the bridge should be secured if possible.

Final Selection. In making the final selection the following principles should be observed as far as practicable.

(*a*) To follow that route which affords the easiest grades. The easiest grade for a given road will depend on the kind of covering adopted for its surface.

(*b*) To connect the places by the shortest and most direct route commensurate with easy grades.

(*c*) To avoid all unnecessary ascents and descents. When a road is encumbered with useless ascents, the wasteful expenditure of power is considerable.

(*d*) To give the center line such a position, with reference to the natural surface of the ground, that the cost of construction shall be reduced to the smallest possible amount.

(*e*) To cross all obstacles (where structures are necessary) as nearly as possible at right angles. The cost of skew structures increases nearly as the square of the secant of the obliquity.

(*f*) To cross ridges through the lowest pass which occurs.

(*g*) To cross either under or over railroads; for grade crossings mean danger to every user of the highway.

Examples of Cases to be Treated. In laying out the line of a road, there are three cases which may have to be treated, and each of these is exemplified in the contour map, Fig. 4. First, the two places to be connected, as the towns A and B on the plan, may be both situated in the same valley, and upon the same side of it; that is, they are not separated from each other by the main stream which drains the valley. This is the simplest case. Secondly, although both in the same valley, the two places may be on opposite sides of the valley, as at A and C, being separated by the main river. Thirdly, they may be situated in different valleys, separated by an intervening ridge of ground more or less elevated, as at A and D. In laying out an extensive line of road, it frequently happens that all these cases have to be dealt with.

The most perfect road is that of which the course is perfectly straight and the surface practically level; and, all other things being the same, the best road is that which answers nearest to this description.

Now, in the first case, that of the two towns situated on the same side of the main valley, there are two methods which may be pursued in forming a communication between them. A road following the direct line between them, shown by the thick dotted line A B, may be made, or a line may be adopted which will gradually and equally incline from one town to another, supposing them to be at different levels; or, if they are on the same level, the line should keep at that level throughout its entire course, following all the sinuosities and curves which the irregular formation of the country may render necessary for the fulfillment of these conditions. According to the first method, a level or uniformly inclined road might be made from one to the other; this line would cross all the valleys and streams which run down to the main river, thus necessitating deep cuttings, heavy embankments, and numerous bridges; or these expensive works might be avoided by following the sinuosities of the valley. When the sides of the main valley are pierced by numerous ravines with projecting spurs and ridges intervening, instead of following the sinuosities, it will be found better to make a nearly straight line cutting through the projecting points in such a way that the material excavated should be just sufficient to fill the hollows.

Of all these, the best is the straight or uniformly inclined, or level road, although at the same time it is the most expensive. If the importance of the traffic passing between the places is not sufficient to warrant so great an outlay, it will become a matter of consideration whether the course of the road should be kept straight, its surface being made to undulate with the natural face of the country; or whether, a level or equally inclined line being adopted, the course of the road should be made to deviate from the direct line, and follow the winding course which such a condition is supposed to necessitate.

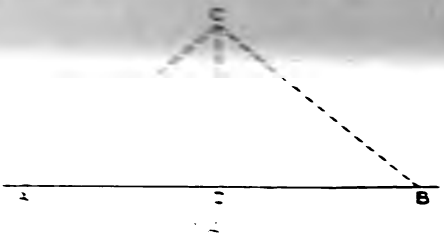
In the second case, that of two places situated on opposite sides of the same valley, there is, in like manner, the choice of a perfectly straight line to connect them, which would probably require a big embankment if the road was kept level, or steep inclines if it followed the surface of the country; or by winding the road, it may be carried across the valley at a higher point, where, if the level road be taken, the embankment would not be so high, or, if kept on the surface, the inclination would be reduced.

In the third case, there is, in like manner, the alternative of carrying the road across the intervening ridge in a perfectly straight line, or of deviating it to the right and left, and crossing the ridge at a point where the elevation is less.

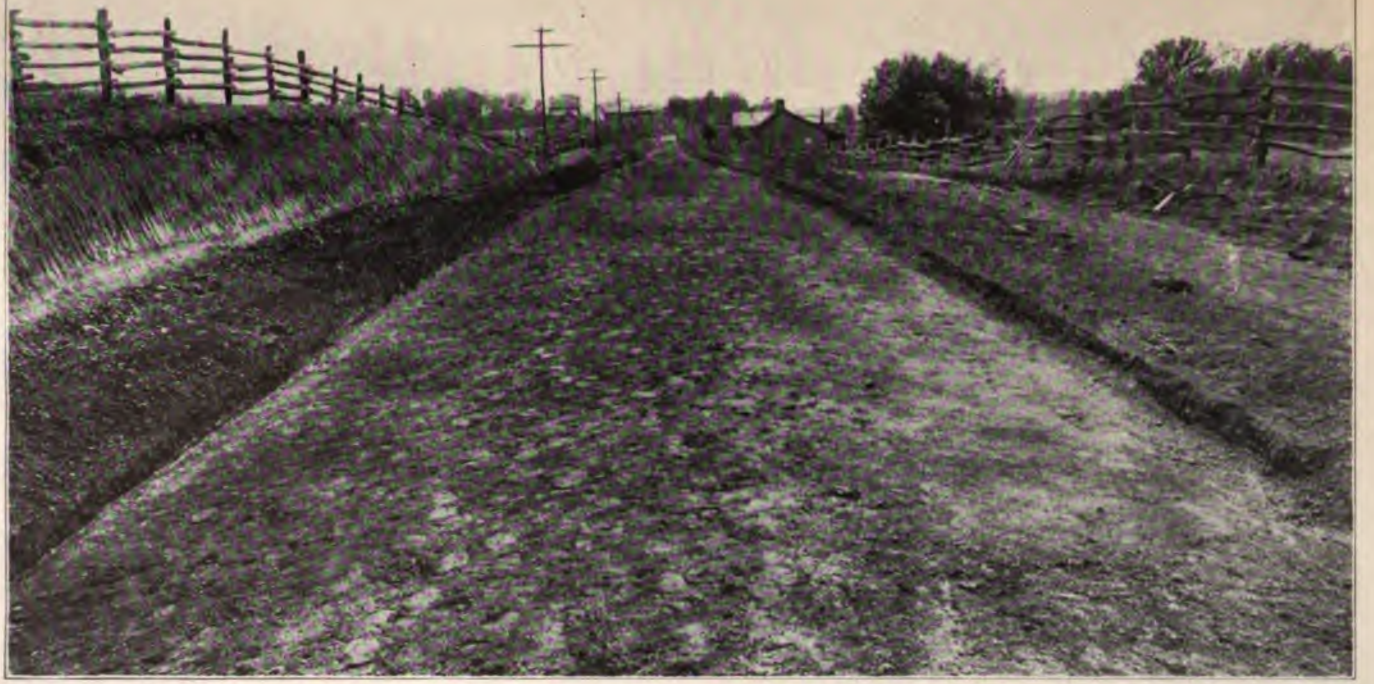
The proper determination of the question which of these courses is the best under certain circumstances involves a consideration of the comparative advantages and disadvantages of inclines and curves. What additional increase in the length of a road would be equivalent to a given inclined plane upon it; or conversely, what inclination might be given to a road as an equivalent to a given decrease in its length?" To satisfy this question, the comparative force required to draw different vehicles with given loads must be known, both upon level and variously inclined roads.

The route which will give the most general satisfaction consists in following the valleys as much as possible and rising afterward by gentle grades. This course traverses the cultivated lands, regions studded with farmhouses and factories. The value of such a line is much more considerable than that of a route by the ridges. The water courses which flow down to the main valley are, it is true, crossed where they are the largest and require works of large dimensions, but also they are fewer in number.

Intermediate Towns. Suppose that it is desired to form a road between two distant towns, A and B, Fig. 6, and let us for the present neglect altogether the consideration of the physical features of the intervening country, assuming that it is equally favorable whichever line we select. Now at first sight, it would appear that under such circumstances a perfectly straight line drawn from one town to the other would be the best that could be chosen. On more careful examination however, of the locality, we may find that there is a third town, C, situated somewhat on one side of the straight line [illegible] from A to B; and although our primary object [illegible] latter, that it would nevertheless be of [illegible] whole of the three towns were put into [illegible] each other.



[illegible] effected in three different ways, any one of which [illegible] instances, be the best. In the first place, we [illegible] as originally suggested, form a straight road from A to B, and in a similar manner two other straight roads from A to C, and from B to C, and this would be the most perfect way of effecting the object in view, the distance between any of the two towns being reduced to the least possible. It would, however, be attended with



ROLLED SUBGRADE OF A LIMESTONE MACADAM ROAD AT JOHNSON CITY, TENNESSEE

Constructed as an object-lesson on road-building, by the Office of Public Roads of the U. S. Department of Agriculture. For other stages in improvement of this highway, see pages 282, 295, and 299.

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considerable expense, and it would be requisite to construct a much greater length of road than according to the second plan, which would be to form, as before, a straight road from A to B, and from C to construct a road which should join the former at a point D, so as to be perpendicular to it. The traffic between A or B and C would proceed to the point D and then turn off to C. With this arrangement, while the length of the roads would be very materially decreased, only a slight increase would be occasioned in the distance between C and the other two towns. The third method would be to form only the two roads A C and C B, in which case the distance between A and B would be somewhat increased, while that between A C or B and C would be diminished, and the total length of road to be constructed would also be lessened.

As a general rule it may be taken that the last of these methods is the best and most convenient for the public; that is to say, that if the physical character of the country does not determine the course of the road, it will generally be found best not to adopt a perfectly straight line, but to vary the line so as to pass through all the principal towns near its general course.

Mountain Roads. The location of roads in mountainous countries presents greater difficulties than in an ordinary undulating country; the same latitude in adopting undulating grades and choice of position is not permissible, for the maximum must be kept before the eye perpetually. A mountain road has to be constructed on the maximum grade or at grades closely approximating it, and but one fixed point can be obtained before commencing the survey, and that is the lowest pass in the mountain range; from this point the survey must be commenced. The reason for this is that the lower slopes of the mountain are flatter than those at their summit; they cover a larger area, and merge into the valley in diverse undulations. So that a road at a foot of a mountain may be carried at will in the desired direction by more than one route, while at the top of a mountain range any deviation from the lowest pass involves increased length of line. The engineer having less command of the ground, owing to the reduced area he has to deal with and the greater abruptness of the slopes, is liable to be frustrated in his attempt to get his line carried in the desired direction.

It is a common practice to run a mountain survey up hill, but this should be avoided. Whenever an acute-angled zigzag is met with on a mountain road near the summit, the inference to be drawn is that the line being carried up hill on reaching the summit was too low and the zigzag was necessary to reach the desired pass. The only remedy in such a case is by a resurvey beginning at the summit and running down hill. This method requires a reversal of that usually adopted. The grade line is first staked out and its horizontal location surveyed afterwards. The most appropriate instrument for this work is a transit with a vertical circle on which the telescope may be set to the angle of the maximum grade.

Loss of Height. Loss of height is to be carefully avoided in a mountain road. By loss of height is meant an intermediate rise in a descending grade. If a descending grade is interrupted by the introduction of an unnecessary ascent, the length of the road will be increased over that due to the continuous grade by the length of the portion of the road intervening between the summit of the rise and the point in the road on a level with that rise—a length which is double that due on the gradient to the height of the rise. For example, if a road descending a mountain rises at some intermediate point to cross over a ridge or spur, and the height ascended amounts to 110 feet before the descent is continued, such a road would be just one mile longer than if the descent had been uninterrupted; for 110 feet is the rise due to a half-mile length at 1:24.

Water on Mountain Roads. Water is needed by the workmen and during the construction of the road; it is also very necessary for the traffic, especially during hot weather; and if the road exceeds 5 miles in length, provision should be made to have it either close to or within easy reach of the road. With a little ingenuity the water from springs above the road, if such exist, can be led down to drinking fountains for men, and to troughs for animals.

In a tropical country it would be a matter for serious consideration if the best line for a mountain road 10 miles in length or upwards, but without water, should not be abandoned in favor of a worse line with a water supply available.

Halting Places. On long lines of mountain roads halting places should be provided at frequent intervals.

Alignment. No rule can be laid down for the alignment of a road; it will depend both upon the character of the traffic on it and upon the "lay of the land." To promote economy of transportation it should be straight; but if straightness is obtained at the expense of easy grades that might have been obtained by deflections and increase in length, it will prove very expensive to the community that uses it.

Where curves are necessary, employ the greatest radius possible and never less than fifty feet. They may be circular or parabolic. The parabolic will be found exceedingly useful for joining tangents of unequal length, and for following contour lines; its curvature being least at its beginning and ending, makes the deviations from a straight line less strongly marked than by a circular arc.

When a curve occurs on an ascent, the grade at that place must be diminished in order to compensate for the additional resistance of the curve.

The width of the wheelway on curves must be increased. This increase should be one-quarter of the width for central angles between 90 and 120 degrees, and one-half for angles between 60 and 90 degrees. Excessive crookedness of alignment is to be avoided, for any unnecessary length causes a constant threefold waste; first, of the interest of the capital expended in making that unnecessary portion; secondly, of the ever recurring expense of repairing it; and thirdly, of the time and labor employed on travelling over it.

The curving road around a hill may be often no longer than the straight one over it, for the latter is straight only with reference to the horizontal plane, while it is curved as to the vertical plane; the former is curved as to the horizontal plane, but straight as to the vertical plane. Both lines curve, and we call the one passing over the hill straight only because its vertical curvature is less apparent to our eyes.

The difference in length between a straight road and one which is slightly curved is very small. If a road between two places ten miles apart were made to curve so that the eye could nowhere see farther than one-quarter of a mile of it at once, its length would exceed that of a straight road between the same points by only about four hundred and fifty feet.

Zigzags. The method of surmounting a height by a series of zigzags or by a series of reaches with practicable curves at the turns, is objectionable.

(1) An acute-angled zigzag obliges the traffic to reverse its direction without affording it convenient room for the purpose. The consequence is that with slow traffic a single train of vehicles is brought to a stand, while if two trains of vehicles travelling in opposite directions meet at the zigzag a block ensues.

(2) With zigzags little progress is made towards the ultimate destination of the road; height is surmounted, but horizontal distance is increased for which there is no necessity or compensation.

(3) Zigzags are dangerous. In case of a runaway down hill the zigzag must prove fatal.

(4) If the drainage cannot be carried clear of the road at the end of each reach, it must be carried under the road in one reach only to appear again at the next, when a second bridge, culvert, or drain will be required, and so on at the other reaches. If the drainage can be carried clear at the termination of each reach, the lengths between the curves will be very short, entailing numerous zigzag curves, which are expensive to construct and maintain.

Final Location. The route being finally determined upon, it requires to be located. This consists in tracing the line, placing a stake at every 100 feet on the straight portions and at every 50 or 25 feet on the curves. At the tangent point of curves, and at points of compound and reverse curves, a larger and more permanent stake should be placed. Lest those stakes should be disturbed in the process of construction, their exact distance from several points outside of the ground to be occupied by the road should be carefully measured and recorded in the notebook, so that they may be replaced. The stakes above referred to show the position of the center line of the road, and form the base line from which all operations of construction are carried on. Levels are taken at each stake, and cross levels are taken at every change of longitudinal slope.

Construction Profile. The construction or working profile is made from the levels obtained on location. It should be drawn to a horizontal scale of 400 feet to the inch and a vertical scale of 20 feet to the inch. Fig. 7 represents a portion of such a profile. The figures in column A represent the elevation of the ground at every 100 feet, or where a stake has been driven, above datum. The figures in column B are the elevations of the grade above datum. The figures in column C indicate the depth of cutting or height of fill; they are obtained by taking the difference between the level of the road and the level of the surface of the ground. The straight line at the top represents the grade of the road; the upper surface of the road when finished would be somewhat higher than this, while the given line represents what is termed the sub-grade or formation level. All the dimensions refer to the formation level, to which the surface of the ground is to be formed to receive the road covering.

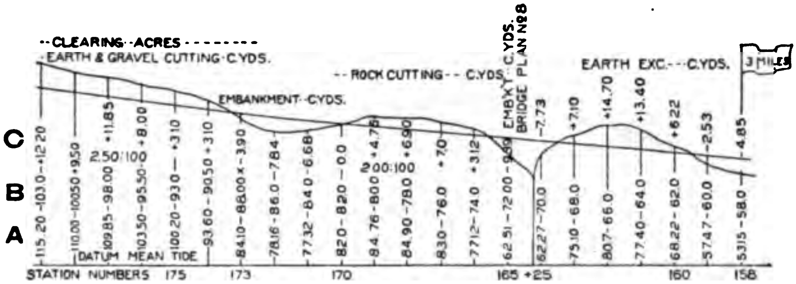

Fig. 7.

At all changes in the rate of inclination of the grade line a heavier vertical line should be drawn.

Gradient. The grade of a line is its longitudinal slope, and is designated by the proportion between its length and the difference of height of its two extremes. The ratio of these two qualities gives it its name; if the road ascends or falls one foot in every twenty feet of its length, it is said to have a grade of 1 : 20 or a 5 per cent grade. Grades are of two kinds, maximum and minimum. The maximum is the steepest which is to be permitted and which on no account is to be exceeded. The minimum is the least allowable for good drainage. (For method of designating grades see Table 9).

Determination of Gradients. The maximum grade is fixed by two considerations, one relating to the power expended in ascending, the other to the acceleration in descending the incline.

There is a certain inclination, depending upon the degree of perfection given to the surface of the road, which cannot be exceeded

without a direct loss of tractive power. This inclination is that in descending which, at a uniform speed, the traces slacken, or which causes the vehicles to press on the horses; the limiting inclination within which this effect does not take place is the angle of repose.

TABLE 9.

American method. Feet per 100 feet.	English method.	Feet per mile.	Angle with the horizon.
¼	1 : 400	13.2	0° 8′ 36″
½	1 : 200	26.4	0 17 11
¾	1 : 150	39.6	0 22 55
1	1 : 100	52.8	0 34 23
1¼	1 : 80	66	0 42 58
1½	1 : 66⅔	79.2	0 51 28
1¾	1 : 57¼	92.4	1 0 51
2	1 : 50	105.6	1 8 6
2¼	1 : 44¼	118.8	1 17 39
2½	1 : 40	132	1 25 57
2¾	1 : 36⅓	145.2	1 34 22
3	1 : 33⅓	158.4	1 43 08
3¼	1 : 30¾	171.6	1 51 42
3½	1 : 28⅗	184.8	2 0 16
3¾	1 : 26⅔	198	2 8 51
4	1 : 25	211.2	1 17 26
4¼	1 : 23½	224.4	2 26 10
4½	1 : 22¼	237.6	2 34 36
4¾	1 : 21	250.8	2 43 35
5	1 : 20	264	2 51 44
6	1 : 13⅔	316.8	3 26 12
7	1 : 14⅔	369.6	4 0 15
8	1 : 12½	422.4	4 34 26
9	1 : 11⅑	475.2	5 8 31
10	1 : 10	528	5 42 37

The angle of repose for any given road surface can be easily ascertained from the tractive force required upon a level with the same character of surface. Thus if the force necessary on a level to overcome the resistance of the load is $\frac{1}{40}$ of its weight, then the same fraction expresses the angle of repose for that surface.

On all inclines less steep than the angle of repose a certain amount of tractive force is necessary in the descent as well as in the ascent, and the mean of the two drawing forces, ascending and descending, is equal to the force along the level of the road. Thus on such inclines, as much mechanical force is gained in the descent as is lost in the ascent. From this it might be inferred that when a vehicle passes alternately each way along the road, no real loss is occasioned by the inclination of the road; such is not, however, practically the fact with animal power, for while it is necessary in the ascending journey to have either a less or a greater number of horses than would be requisite if the road were entirely level, no corresponding reduction can be made in the descending journey. On inclines which are more steep than the angle of repose, the load presses on the horses during their descent, so as to impede their action, and their power is expended in checking the descent of the load; or if this effect be prevented by the use of any form of drag or brake, then the power expended on such a drag or brake corresponds to an equal quantity of mechanical power expended in the ascent, for which no equivalent is obtained in the descent.

The maximum grade for a given road will depend (1) upon the class of traffic that will use it, whether fast and light, slow and heavy, or mixed, consisting of both light and heavy; (2) upon the character of the pavement adopted; and (3) upon the question of cost of construction. Economy of motive power and low cost of construction are antagonistic to each other, and the engineer will have to weigh the two in the balance.

For fast and light traffic the grades should not exceed 2 per cent; for mixed traffic 3 per cent may be adopted; while for slow traffic combined with economy 5 per cent should not be exceeded. This grade is practicable but not convenient.

Minimum Grade. From the previous considerations it would appear that an absolutely level road was the one to be sought for, but this is not so; there is a minimum or least allowable grade which the road must not fall short of, as well as a maximum one which it must not exceed. If the road was perfectly level in its longitudinal direction, its surface could not be kept free from water without giving it so great a rise in its middle as would expose vehicles to the danger of overturning. The minimum grade commonly used is 1 per cent.

Undulating Grades. From the fact that the power required to move a load at a given velocity on a level road is decreased on a descending grade to the same extent it is increased in ascending the same grade, it must not be inferred that the animal force expended in passing alternately each way over a rising and falling road will gain as much in descending the several inclines as it will lose in ascending them. Such is not the case. The animal force must be sufficient, either in power or number, to draw the load over the level portions and up the steepest inclines of the road, and in practice no reduction in the number of horses can be made to correspond with the decreased power required in descending the inclines.

The popular theory that a gentle undulating road is less fatiguing to horses than one which is perfectly level is erroneous. The assertion that the alternations of ascent, descent, and levels call into play different muscles, allowing some to rest while others are exerted, and thus relieving each in turn, is demonstrably false, and contradicted by the anatomical structure of the horse. Since this doctrine is a mere popular error, it should be utterly rejected, not only because false in itself, but still more because it encourages the building of undulating roads, and this increases the labor and cost of transportation upon them.

Level Stretches. On long ascents it is generally recommended to introduce level or nearly level stretches at frequent intervals in order to rest the animals. These are objectionable when they cause loss of height, and animals will be more rested by halting and unharnessing for half an hour than by travelling over a level portion. The only case which justifies the introduction of levels into an ascending road is where such levels will advance the road towards its objective point; where this is the case there will be no loss of either length or height, and it will simply be exchanging a level road below for a level road above.

Establishing the Grade. When the profile of a proposed route has been made, a grade line is drawn upon it (usually in red) in such a manner as to follow its general slope, but to average its irregular elevation and depressions.

If the ratio between the whole distance and the height of the line is less than the maximum grade intended to be used, this line will be satisfactory; but if it be found steeper, the cuttings or the length of the line will have to be increased; the latter is generally preferable.

The apex or meeting point of all curves should be rounded off by a vertical curve, as shown in Fig. 8, thus slightly changing the grade at and near the point of intersection. A vertical curve rarely need extend more than 200 feet each way from that point.

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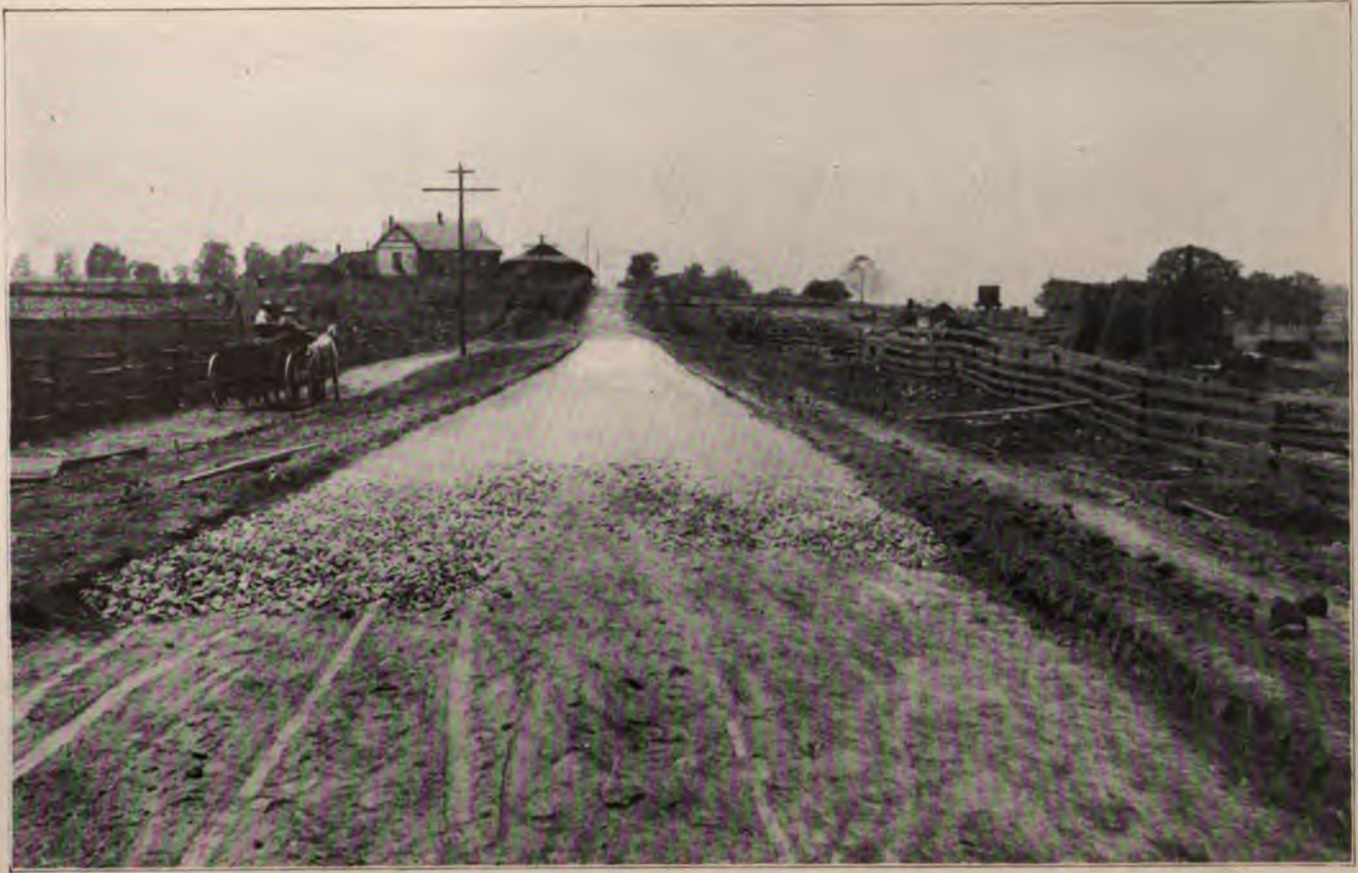


ROLLED FIRST COURSE IN CONSTRUCTION OF A LIMESTONE MACADAMIZED ROAD AT JOHNSON CITY, TENNESSEE

This improvement was carried out by the Office of Public Roads of the U. S. Department of Agriculture, as an object-lesson on road-building. For other stages in the work of construction, see pages 282, 286, and 299.

Let A B, B C, be two grades in profile, intersecting at station B, and let A and C be the adjacent stations. It is required to join the grades by a vertical curve extending from A to C. Imagine a chord drawn from A to C. The elevation of the middle point of the chord will be a mean of the elevations of the grade at A and C, and one-half of the difference between this and the elevation of the grade at B will be the middle ordinate of the curve. Hence we have

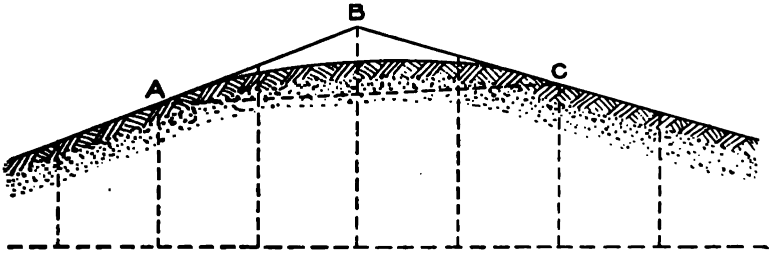


Fig. 8.

$$M = \frac{1}{2}\left(\frac{\text{grade A} + \text{grade C}}{2} - \text{grade B}\right),$$

in which M equals the correction in grade for the point B. The correction for any other point is proportional to the square of its distance from A or C. Thus the correction A + 25 is $\frac{1}{16}$ M; at A + 50 it is $\frac{1}{4}$ M; at A + 75 it is $\frac{9}{16}$ M; and the same for corresponding points on the other side of B. The corrections in this case shown are subtractive, since M is negative. They are additive when M is positive, and the curve concave upward.

WIDTH AND TRANSVERSE CONTOUR.

A road should be wide enough to accommodate the traffic for which it is intended, and should comprise a wheelway for vehicles and a space on each side for pedestrians.

The wheelway of country highways need be no wider than is absolutely necessary to accommodate the traffic using it; in many places a track wide enough for a single team is all that is necessary. But the breadth of the land appropriated for highway purposes should be sufficient to provide for all future increase of traffic. The wheelways of roads in rural sections should be double; that is, one portion paved (preferably the center), and the other left with the natural soil. The latter if kept in repair will for at least one-half the year be preferred by teamsters.

The minimum width of the paved portion, if intended to carry two lines of travel, is fixed by the width required to allow two vehicles to pass each other safely. This width is 16 feet. If intended for a single line of travel, 8 feet is sufficient, but suitable turnouts must be provided at frequent intervals. The most economical width for any roadway is some multiple of eight.

Wide roads are the best; they expose a larger surface to the drying action of the sun and wind, and require less supervision than narrow ones. Their first cost is greater than narrow ones, and that nearly in the ratio of the increased width.

The cost of maintaining a mile of road depends more upon the extent of the traffic than upon the extent of its surface, and unless extremes be taken, the same quantity of material will be necessary for the repair of the road whether wide or narrow, which is subjected to the same amount of traffic. The cost of spreading the materials over the wide road will be somewhat greater, but the cost of the materials will be the same. On narrow roads the traffic, being confined to one track, will wear more severely than if spread over a wider surface.

The width of land appropriated for road purposes varies in the United States from 49½ feet to 66 feet; in England and France from 26 to 66 feet. And the width or space macadamized is also subject to variation; in the United States the average width is 16 feet; in France it varies between 16 and 22 feet; in Belgium 8¼ feet seems to be the regular width, while in Austria from 14½ to 26¼ feet.

Transverse Contour. The center of all roadways should be higher than the sides. The object of this is to facilitate the flow of the rain water to the gutters. Where a good surface is maintained a very moderate amount of rise is sufficient for this purpose. Earth roads require the most and asphalt the least. The rise should bear a certain proportion to the width of the carriageway. The most suitable proportions for the different paving materials is shown in table 10.

Form of Transverse Contour. All authorities agree that the form should be convex, but they differ in the amount and form of the convexity. Circular arcs, two straight lines joined by a circular arc, and ellipses, all have their advocates.

TABLE 10.

Kind of Surface.		Proportions of the Carriageway. Width.
Earth	Rise at center	$\frac{1}{40}$
Gravel	" " "	$\frac{1}{50}$
Broken Stone	" " "	$\frac{1}{60}$

For country roads a curve of suitable convexity may be obtained as follows: Give $\frac{7}{8}$ of the total rise at $\frac{1}{4}$ the width from the center to the side, and $\frac{5}{8}$ of the total rise at $\frac{1}{2}$ the width (Fig. 9).

Excessive height and convexity of cross-section contract the width of the wheelway, by concentrating the traffic at the center, that being the only part where a vehicle can run upright. The force required to haul vehicles over such cross-sections is increased, because an undue proportion of the load is thrown upon two wheels instead of being distributed equally over the four. The continual tread of horses' feet in one track soon forms a depression which holds water, and the surface is not so dry as with a flat section, which allows the traffic to distribute itself over the whole width.

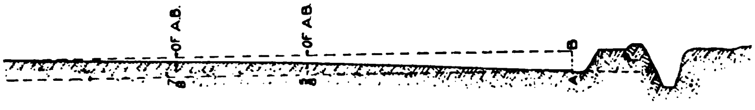


Fig. 9.

Sides formed of straight lines are also objectionable. They wear hollow, retain water, and defeat the object sought by raising the center.

The required convexity should be obtained by rounding the formation surface, and not by diminishing the thickness of the covering at the sides.

Although on hillside and mountain roads it is generally recommended that the surface should consist of a single slope inclining inwards, there is no reason for or advantage gained by this method. The form best adapted to these roads is the same as for a road under ordinary conditions.

With a roadway raised in the center and the rain water draining off to gutters on each side, the drainage will be more effectual and speedy than if the drainage of the outer half of the road has to pass over the inner half. The inner half of such a road is usually subjected to more traffic than the outer half. If formed of a straight incline, this side will be worn hollow and retain water. The inclined flat section never can be properly repaired to withstand the traffic. Consequently it never can be kept in good order, no matter how constantly it may be mended. It is always below par and when heavy rain falls it is seriously damaged.

DRAINAGE.

In the construction of roads, drainage is of the first importance. The ability of earth to sustain a load depends in a large measure upon the amount of moisture retained by it. Most earths form a good firm foundation so long as they are kept dry, but when wet they lose their sustaining power, becoming soft and incoherent.

The drainage of roadways is of two kinds, viz., surface and subsurface. The first provides for the speedy removal of all water falling on the surface of the road; the second provides for the removal of the underground water found in the body of the road, a thorough removal of which is of the utmost importance and essential to the life of the road. A road covering placed on a wet undrained bottom will be destroyed by both water and frost, and will always be troublesome and expensive to maintain; perfect subsoil drainage is a necessity and will be found economical in the end even if in securing it considerable expense is required.

The methods employed for securing the subsoil drainage must be varied according to the character of the natural soil, each kind of soil requiring different treatment.

The natural soil may be divided into the following classes: silicious, argillaceous, and calcareous; rock, swamps, and morasses.

The silicious and calcareous soils, the sandy loams and rock, present no great difficulty in securing a dry and solid foundation. Ordinarily they are not retentive of water and therefore require no underdrains; ditches on each side of the road will generally be found sufficient.

The argillaceous soils and softer marls require more care; they retain water and are difficult to compact, except at the surface; and they are very unstable under the action of water and trost.

FINISHED LIMESTONE MACADAM ROAD AT JOHNSON CITY, TENNESSEE

Constructed by the Office of Public Roads, U. S. Department of Agriculture, as an object-lesson on road-building. For various stages in the construction of this road, see pages 282, 286, and 295.

The drainage of these soils may be effected by transverse drains and deep side ditches of ample width. The transverse drains are placed across the road, not at right angles but in the form of an inverted V with the point directed up hill; the depth at the angle point should not be less than 18 inches below the subgrade surface, and each branch should descend from the apex to the side ditches with a fall of not less than 1 inch in 5 feet. The distance apart of these drains will depend upon the wetness of the soil; in the case of very wet soil they should be at intervals of 15 feet, which may be increased to 25 feet as the ground becomes drier and firmer.

The transverse drains are best formed of unglazed circular tile of a diameter not less than 3 inches, jointed with loose collars. The tiles are made from terra cotta or burnt clay, are porous, and are superior to all other kinds of drains. They carry off the water with greater ease, rarely if ever get choked up, and only require a slight inclination to keep the water moving through them.

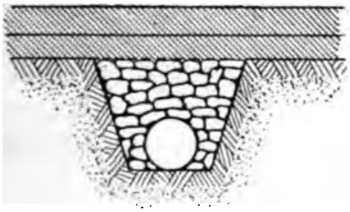


Fig. 10. Tile Drain.

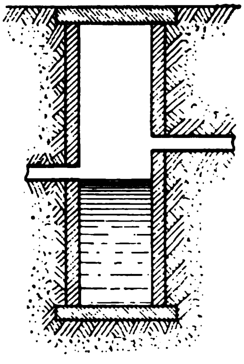


Fig. 11. Silt Basin.

The tiles are made in a variety of forms, as horseshoe, sole, double sole, and round, the name being derived from the shape of the cross-sections. Round tile is superior to all other forms. The inside diameter of these tiles varies from 1¼ to 6 inches, but they are manufactured as large as 24 inches. Pieces of the larger pipe serve as collars for the smaller ones. They are made in lengths of 12, 14 and 24 inches, and in thickness of shell from ¼ of an inch to 1 inch.

The collar which encircles the joint of the small tile allows a large opening, and at the same time prevents sand and silt from entering the drain. Perishable material should not be used for jointing. When laid in the ditch they should be held in place by small stones. Connections should be made by proper Y-branches.

The outlets may be formed by building a dwarf wall of brick or stone, whichever is the cheapest or most convenient in the locality. The outlet should be covered with an iron grating to prevent vermin entering the drain pipes, building nests and thus choking up the waterway. (See Fig. 12.)

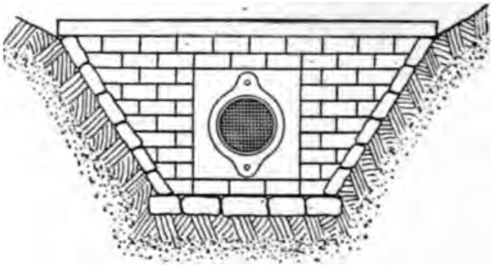


Fig. 12. Outlet.

Silt-basins should be constructed at all junctions and wherever else they may be considered necessary; they may be made from a single 6-inch pipe (Fig. 11) or constructed of brick masonry.

The trenches for the tile should be excavated at least 3 feet wide on top and 12 inches on the bottom. After the tiles are laid the trenches must be filled to subgrade level with round field or cobble stones; stones with angular edges are unsuitable for this purpose. Fine gravel, sand, or soil should not be placed over the drains. Bricks and flat stones may be substituted for the tiles, and the trenches filled as above stated.

As tile drains are more liable to injury from frost than those of either brick or stone, their ends at the side ditches should not in very cold climates be exposed directly to the weather, but may terminate in blind drains, or a few lengths of vitrified clay pipe reaching under the road a distance of about 3 to 4 feet from the inner slope of the ditch.

Another method of draining the roadbed offering security from frost is by one or more rows of longitudinal drains. These drains are placed at equal distances from the side ditches and from each other, and discharge into cross drains placed from 250 to 300 feet apart, more or less, depending on the contour of the ground. The cross drains into which they discharge should be of ample dimensions. On these longitudinal lines of tiles the introduction of catch basins at intervals of 50 feet will facilitate the removal of the water. These catch basins may be excavated three or more feet square and as deep as the tiles are laid. After the tiles are laid the pit is filled with gravel and small stones.

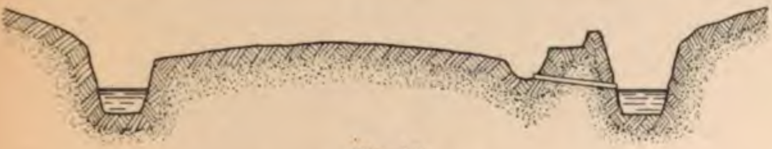


Fig. 13.

Fall of Drains. It is a mistake to give too much fall to small drains, the only effect of which is to produce such a current through them as will wash away or undermine the ground around them, and ultimately cause their own destruction. When a drain is once closed by any obstruction no amount of fall which could be given it will again clear the passage. A drain with a considerable current through it is much more likely to be stopped from foreign matter carried into it, which a less rapid stream could not have transported.

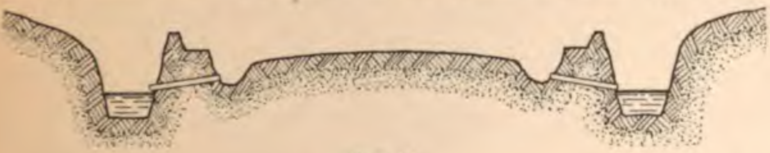


Fig. 14.

A fall of 1 inch in 5 feet will generally be sufficient, and 1 inch in 30 inches should never be exceeded.

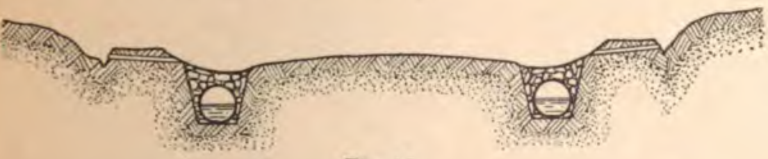


Fig. 15.

Side Ditches are provided to carry away the subsoil water from the base of the road, and the rain water which falls upon its surface; to do this speedily they must have capacity and inclination proportionate to the amount of water reaching them. The width of the bed should not be less than 18 inches; the depth will vary with circumstances, but should be such that the water surface shall not reach the subgrade, but remain at least 12 inches below the crown of the road. The sides should slope at least 1½ to 1.

The longitudinal inclination of the ditch follows the configuration of the general topography, that is, the lines of natural drainage. When the latter has to be aided artificially, grades from 1 in 300 to 1 in 900 will usually answer.

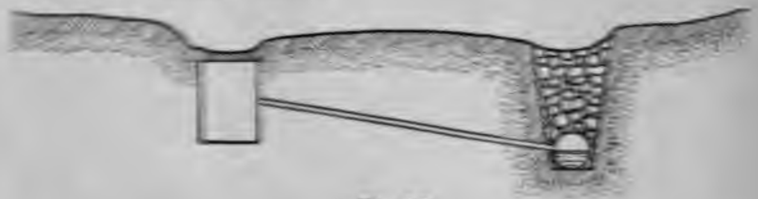


Fig. 16.

In absorbent soil less fall is sufficient, and in certain cases level ditches are permissible. The slopes of the ditches must be protected where the grade is considerable. This can be accomplished by sod revetments, riprapping, or paving.

These ditches may be placed either on the road or land side of the fence. In localities where open ditches are undesirable they may be constructed as shown in Figs. 13 to 17, and may be formed of stone

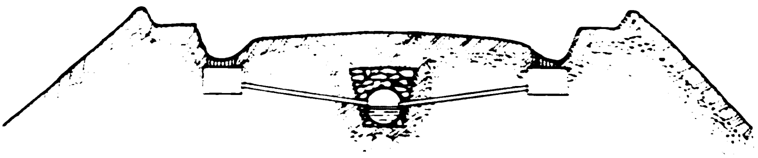


Fig. 17.

or tile pipe, according to the availability of either material. If for any reason two can not be built, build one.

Springs found in the roadbed should be tapped and led into the side ditches.

Drainage of the Surface. The drainage of the roadway surface depends upon the preservation of the cross-section, with regular and uninterrupted fall to the sides, without hollows or ruts in which the water can lie, and also upon the longitudinal fall of the road. If this is not sufficient the road becomes flooded during heavy rainstorms and melting snow, and is considerably damaged.

The removal of surface water from country roads may be effected by the side ditches, into which, when there are no sidewalks, the water flows directly. When there are sidewalks, gutters are formed between the roadway and footpath, as shown in Figs. 13 to 17, and the water is conducted from these gutters into the side ditches by tile pipes laid under the walks at intervals of about 50 feet. The entrance to these pipes should be protected against washing by a rough stone paving. In the case of covered ditches under the footpath the water must be led into them by first passing through a catch basin. These are small masonry vaults covered with iron gratings to prevent the ingress of stones, leaves, etc. Connection from the catch basin is made by a tile pipe about 6 inches in diameter. The mouth of this pipe is placed a few feet above the bottom of the catch basin, and the space below it acts as a depository for the silt carried by the water, and is cleaned out periodically. The catch basins may be placed from 200 to 300 feet apart. They should be made of dimensions sufficient to convey the amount of water which is liable to flow into them during heavy and continuous rains.

If on inclines the velocity of the water is greater than the nature of the soil will withstand, the gutters will be roughly paved. In all cases, the slope adjoining the footpath should be covered with sod.

A velocity of 30 feet a minute will not disturb clay with sand and stone. 40 feet per minute will move coarse sand. 60 feet a minute will move gravel. 120 feet a minute should move round pebbles 1 inch in diameter, and 180 feet a minute will move angular stones 1$\frac{3}{4}$ inches in diameter.

The scour in the gutters on inclines may be prevented by small weirs of stones or fascines constructed by the roadmen at a nominal cost. At junctions and crossroads the gutters and side ditches require careful arrangement so that the water from one road may not be thrown upon another; cross drains and culverts will be required at such places.

Water Breaks to turn the surface drainage into the side ditches should not be constructed on improved roads. They increase the grade and are an impediment to convenient and easy travel. Where it is necessary that water should cross the road a culvert should be built.

On the side hill or mountain roads catch-water ditches should be cut on the mountain side above the road, to cut off and convey the drainage of the ground above them to the neighboring ravines. The size of these ditches will be determined by the amount of rainfall, extent of drainage from the mountain which they intercept, and by the distances of the ravine water courses on each side.

The inner road gutter should be of ample dimensions to carry off the water reaching it; when in soil, it should be roughly paved with stone. When paving is not absolutely necessary, but it is desirable to arrest the scouring action of running water during heavy rains, stone weirs may be erected across the gutter at convenient intervals. The outer gutter need not be more than 12 inches wide and 9 inches deep. The gutter is formed by a depression in the surface of the road close to the parapet or revetted earthen protection mound. The drainage which falls into this gutter is led off through the parapet, or other roadside protection at frequent intervals. The guard stones on the outside of the road are placed in and across this gutter, just below the drainage holes, so as to turn the current of the drainage into these holes or channels. On straight reaches, with parapet protection, drainage holes with guard stones should be placed every 20 feet apart. Where earthen mounds are used and it may not be convenient to have the drainage holes or channels every 20 feet, the guardstones are to be placed in advance of the gutter to allow the drainage to pass behind them. This drainage is either to be run off at the cross drainage of the road, or to be turned off as before by a guard stone set across the gutter.

At re-entering turns, where the outer side of the road requires particular protection, guard stones should be placed every 4 feet. As all re-entering turns should be protected by parapets, the drainage holes through them may be placed as close together as desired.

Culverts are necessary for carrying under a road the streams it crosses, and also for conveying the surface water collected in the side ditches from the upper side to that side on which the natural water courses lie.

Especial care is required to provide an ample way for the water to be passed. If the culvert is too small, it is liable to cause a washout, entailing interruption of traffic and cost of repairs, and possibly may cause accidents that will require payment of large sums for damages. On the other hand, if the culvert is made unnecessarily large, the cost of construction is needlessly increased.

The area of waterway required depends (1) upon the rate of rainfall; (2) the kind and condition of the soil; (3) the character and inclination of the surface; (4) the condition and inclination of the bed of the stream; (5) the shape of the area to be drained, and the position of the branches of the stream; (6) the form of the mouth and the inclination of the bed of the culvert; and (7) whether it is permissible to back the water up above the culvert, thereby causing it to discharge under a head.

(1) It is the maximum rate of rainfall during the severest storms which is required in this connection. This varies greatly in different sections of the country.

The maximum rainfall as shown by statistics is about one inch per hour (except during heavy storms), equal to 3,630 cubic feet per acre. Owing to various causes, not more than 50 to 75 per cent of this amount will reach the culvert within the same hour.

Inches of rainfall × 3,630 = cubic feet per acre.

Inches of rainfall × 2,323,200 = cubic feet per square mile.

(2) The amount of water to be drained off will depend upon the permeability of the surface of the ground, which will vary greatly with the kind of soil, the degree of saturation, the condition of the cultivation, the amount of vegetation, etc.

(3) The rapidity with which the water will reach the water course depends upon whether the surface is rough or smooth, steep or flat, barren or covered with vegetation, etc.

(4) The rapidity with which the water will reach the culvert depends upon whether there is a well-defined and unobstructed channel, or whether the water finds its way in a broad thin sheet. It the water course is unobstructed and has a considerable inclination, the water may arrive at the culvert nearly as rapidly as it falls; but if the channel is obstructed, the water may be much longer in passing the culvert than in falling.

(5) The area of waterway depends upon the amount of the area

to be drained; but in many cases the shape of this area and the position of the branches of the stream are of more importance than the amount of the territory. For example, if the area is long and narrow, the water from the lower portion may pass through the culvert before that from the upper end arrives; or, on the other hand, if the upper end of the area is steeper than the lower, the water from the former may arrive simultaneously with that from the latter. Again, if the lower part of the area is better supplied with branches than the upper portion, the water from the former will be carried past the culvert before the arrival of that from the latter; or, on the other hand, if the upper part is better supplied with branch water courses than the lower, the water from the whole area may arrive at the culvert at nearly the same time. In large areas the shape of the area and the position of the water courses are very important considerations.

(6) The efficiency of a culvert may be very materially increased by so arranging the upper end that the water may enter into it without being retarded. The discharging capacity of a culvert can be greatly increased by increasing the inclination of its bed, provided the channel below will allow the water to flow away freely after having passed the culvert.

(7) The discharging capacity of a culvert can be greatly increased by allowing the water to dam up above it. A culvert will discharge twice as much under a head of four feet as under a head of one foot. This can be done safely only with a well constructed culvert.

The determination of the values of the different factors entering into the problem is almost wholly a matter of judgment. An estimate for any one of the above factors is liable to be in error from 100 to 200 per cent, or even more, and of course any result deduced from such data must be very uncertain. Fortunately, mathematical exactness is not required by the problem nor warranted by the data. The question is not one of 10 or 20 per cent of increase; for if a 2-foot pipe is sufficient, a 3-foot pipe will probably be the next size, an increase of 225 per cent; and if a 6-foot arch culvert is too small, an 8-foot will be used, an increase of 180 per cent. The real question is whether a 2-foot pipe or an 8-foot arch culvert is needed.

Valuable data on the proper size of any particular culvert may be obtained (1) by observing the existing openings on the same stream; (2) by measuring, preferably at time of high water, a cross-section of the stream at some narrow place; and (3) determining the height of high water as indicated by drift and the evidence of the inhabitants of the neighborhood.

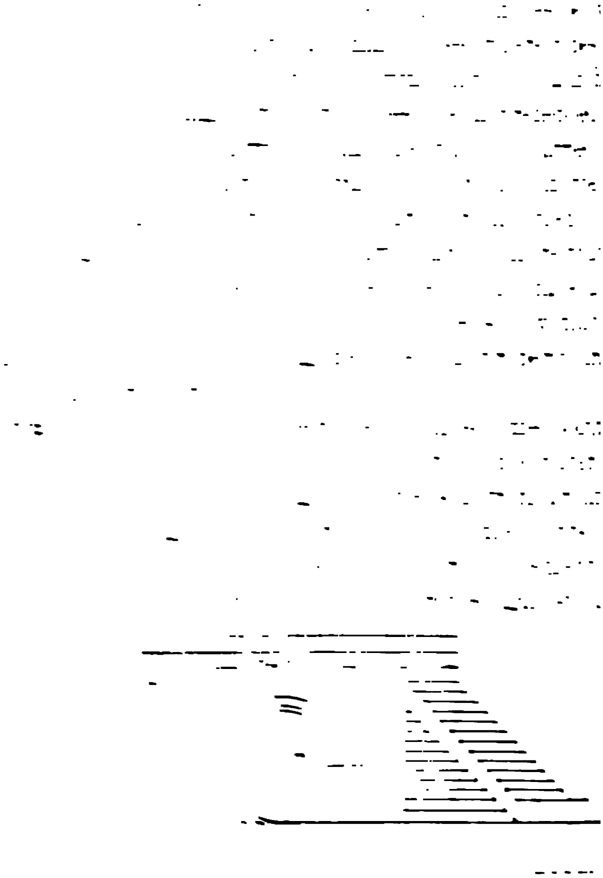
On mountain roads or roads subjected to heavy rainfall culverts of ample dimensions should be provided wherever required, and it will be more economical to construct them of masonry. In localities where boulders and other debris are likely to be washed down during wet weather, it will be a good precaution to construct catch pools at the entrance of all culverts and cross drains for the reception of such matter. In hard soil or rock these catch pools will be simple well-like excavations, with their bottom two or three feet below the entrance sill or floor of the culvert or drain. Where the soil is soft they should be lined with stone laid dry; if very soft, with masonry. The size of the catch pools will depend upon the width of the drainage works. They should be wide enough to prevent the drains from being injured by falling rocks and stones of a not inordinate size.

The use of catch pools obviates the necessity of building culverts and drains at an angle to the axis of the road. Oblique structures are objectionable, as being longer than if set at right angles and by reason of the acute- and obtuse-angled terminations to their piers, abutments, and coverings.

Materials for Culverts. Culverts may be of stone, brick, vitrified earthenware, or iron pipe. Wood should be absolutely avoided.

For small streams and a limited surface of rainfall either class of pipes, in sizes varying from 12 to 24 inches in diameter, will serve excellently. They are easily laid, and if properly bedded, with the earth tamped about them, are very permanent. Their upper surface should be at least 18 inches below the road surface, and the upper end should be protected with stone paving so arranged that the water can in no case work in around the pipe.

When the flow of water is estimated to be too great for two lines of 24-inch pipes, a culvert is required. If stone abounds, it may be built of large roughly squared stones laid either dry or in mortar. When the span required is more than 5 feet, arch culverts either of stone or brick masonry may be employed. For spans above 15 feet the structure required becomes a bridge.



Although often omitted, the end sections should be protected with a masonry or timber bulkhead. The foundation of the bulkhead should be deep enough not to be disturbed by frost. In constructing the end wall, it is well to increase the fall near the outlet to allow for a possible settlement of the interior sections. When stone and brick abutments are too expensive, a fair substitute can be made by setting posts in the ground and spiking plank to them. When planks are used, it is best to set them with considerable inclination towards the roadbed to prevent their being crowded outward by the pressure of the embankment. The upper end of the culvert should be so protected that the water will not readily find its way along the outside of the pipes, in case the mouth of the culvert should become submerged.

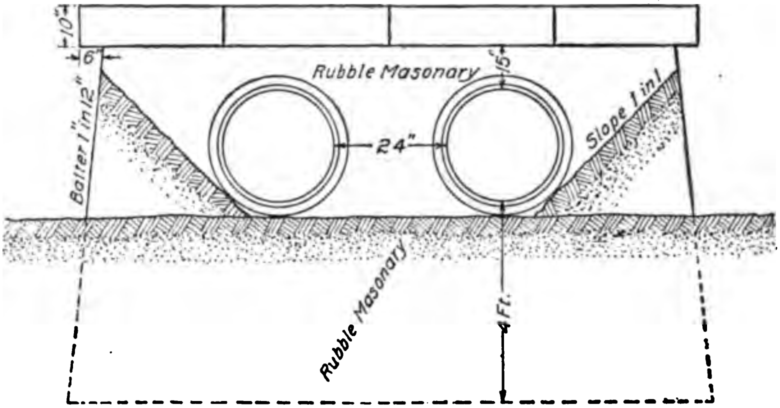


Fig. 19.

When the capacity of one pipe is not sufficient, two or more may be laid side by side as shown in Fig. 19. Although the two small pipes do not have as much discharging capacity as a single large one of equal cross-section, yet there is an advantage in laying two small ones side by side, since the water need not rise so high to utilize the full capacity of the two pipes as would be necessary to discharge itself through a single one of large size.

Iron Pipe Culverts. During recent years iron pipe has been used for culverts on many prominent railroads, and may be used on roads in sections where other materials are unavailable.

In constructing a culvert with cast-iron pipe the points requiring particular attention are (1) tamping the soil tightly around the pipe to prevent the water from forming a channel along the outside, and (2) protecting the ends by suitable head walls and, when necessary, laying riprap at the lower end. The amount of masonry required for the end walls depends upon the relative width of the embankment and the number of sections of pipe used. For example, if the embankment is, say, 40 feet wide at the base, the culvert may consist of three 12-foot lengths of pipe and a light end wall near the toe of the bank; but if the embankment is, say, 32 feet wide, the culvert may consist of two 12-foot lengths of pipe and a comparatively heavy end wall well back from the toe of the bank. The smaller sizes of pipe usually come in 12-foot lengths, but sometimes a few 6-foot lengths are included for use in adjusting the length of the culvert to the width of the bank. The larger sizes are generally 6 feet long.

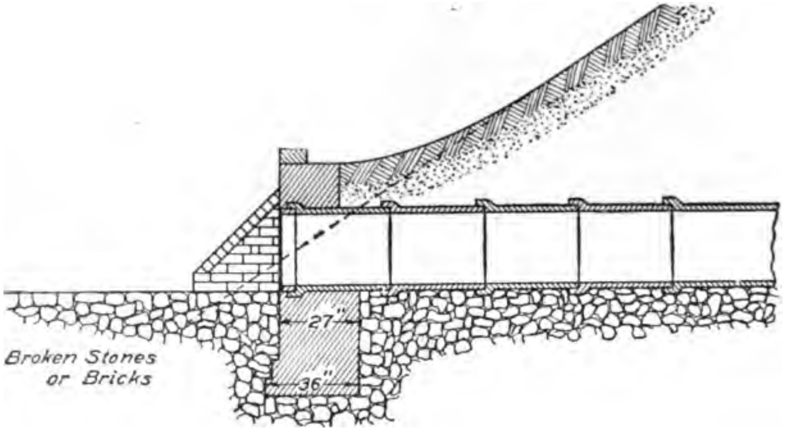


Fig. 20. Section of Pipe Culvert

EARTHWORK.

The term "earthwork" is applied to all the operations performed in the making of excavation and embankments. In its widest sense it comprehends work in rock as well as in the looser materials of the earth's crust.

Balancing Cuts and Fills. In the construction of new roads, the formation of the roadbed consists in bringing the surface of the ground to the adopted grade This grade should be established so as to reduce the earthwork to the least possible amount, both to render the cost of construction low, and to avoid unnecessary marring the appearance of the country in the vicinity of the road. The most desirable position of the grade line is usually that which makes the amount of cutting and filling equal to each other, for any surplus embankment over cutting must be made up by borrowing, and surplus cutting must be wasted, both of these operations involving additional cost for labor and land.

Inclination of Side Slopes. The proper inclination for the side slopes of cutting and embankments depends upon the nature of the soil, the action of the atmosphere and of internal moisture upon it. For economy the inclination should be as steep as the nature of the soil will permit.

The usual slopes in cuttings are:

Solid rock	1 to 1
Earth and Gravel	3½ to 1
Clay	3 or 6 to 1
Fine sand	2 or 3 to 1

The slopes of embankment are usually made 1½ to 1.

Form of Side Slopes. The natural, strongest, and ultimate form of earth slopes is a concave curve, in which the flattest portion is at the bottom. This form is very rarely given to the slopes in constructing them; in fact, the reverse is often the case, the slopes being made convex, thus saving excavation by the contractor and inviting slips.

In cuttings exceeding 10 feet in depth the forming of concave slopes will materially aid in preventing slips, and in any case they will reduce the amount of material which will eventually have to be removed when cleaning up. Straight or convex slopes will continue to slip until the natural form is attained.

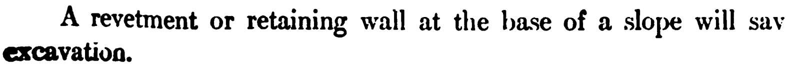


Fig. 21. Cross-Section for Embankment.

A revetment or retaining wall at the base of a slope will save excavation.

In excavations of considerable depth, and particularly in soils liable to slips, the slope may be formed in terraces, the horizontal offsets or benches being made a few feet in width with a ditch on the inner side to receive the surface water from the portion of the side slope above them. These benches catch and retain earth that may fall from the slopes above them. The correct forms for the slopes of embankment and excavation are shown in Figs. 21 and 22.

Covering of Slopes. It is not usual to employ any artificial means to protect the surface of the side slopes from the action of the weather; but it is a precaution which in the end will save much labor and expense in keeping the roadways in good order. The simplest means which can be used for this purpose consists in covering the slopes with good sods, or else with a layer of vegetable mould about four inches thick, carefully laid and sown with grass seed. These means are amply sufficient to protect the side slopes from injury when they are not exposed to any other cause of deterioration than the wash of the rain and the action of frost on the ordinary moisture retained by the soil.

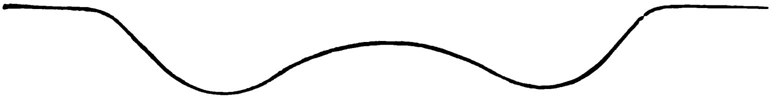


Fig. 22. Cross-Section for Excavation.

A covering of brushwood or a thatch of straw may also be used with good effect; but from their perishable nature they will require frequent renewal and repairs.

Where stone is abundant a small wall of stone laid dry may be constructed at the foot of the slopes to prevent any wash from them being carried into the ditches.

Shrinkage of Earthwork. All materials when excavated increase in bulk, but after being deposited in banks subside or shrink (rock excepted) until they occupy less space than in the pit from which excavated.

Rock, on the other hand, increases in volume by being broken up, and does not settle again into less than its original bulk. The increase may be taken at 50 per cent.

The shrinkage in the different materials is about as follows:

Gravel	8	per cent
Gravel and sand	9	" "
Clay and clay earths	10	" "
Loam and light sandy earths	12	" "
Loose vegetable soil	15	" "
Puddled clay	25	" "

Thus an excavation of loam measuring 1,000 cubic yards will form only about 880 cubic yards of embankment, or an embankment of 1,000 cubic yards will require about 1,120 cubic yards measured in excavation to make it. A rock excavation measuring 1,000 yards will make from 1,500 to 1,700 cubic yards of embankment, depending upon the size of the fragments.

The lineal settlement of earth embankments will be about in the ratio given above; therefore either the contractor should be instructed in setting his poles to guide him as to the height of grade on an earth embankment to add the required percentage to the fill marked on the stakes, or the percentage may be included in the fill marked on the stakes. In rock embankments this is not necessary.

Classification of Earthwork. Excavation is usually classified under the heads *earth*, *hardpan*, *loose rock*, and *solid rock*. For each of these classes a specific price is usually agreed upon, and an extra allowance is sometimes made when the haul or distance to which the excavated material is moved exceeds a given amount.

The characteristics which determine the classes to which a given material belongs are usually described with clearness in the specifications, as:

Earth will include loam, clay, sand, and loose gravel.

Hardpan will include cemented gravel, slate, cobbles, and boulders containing less than one cubic foot, and all other matters of an earthy nature, however compact they may be.

Loose rock will include shale, decomposed rock, boulders, and detached masses of rock containing not less than three cubic feet, and all other matters of a rock nature which may be loosened with a pick, although blasting may be resorted to in order to expedite the work.

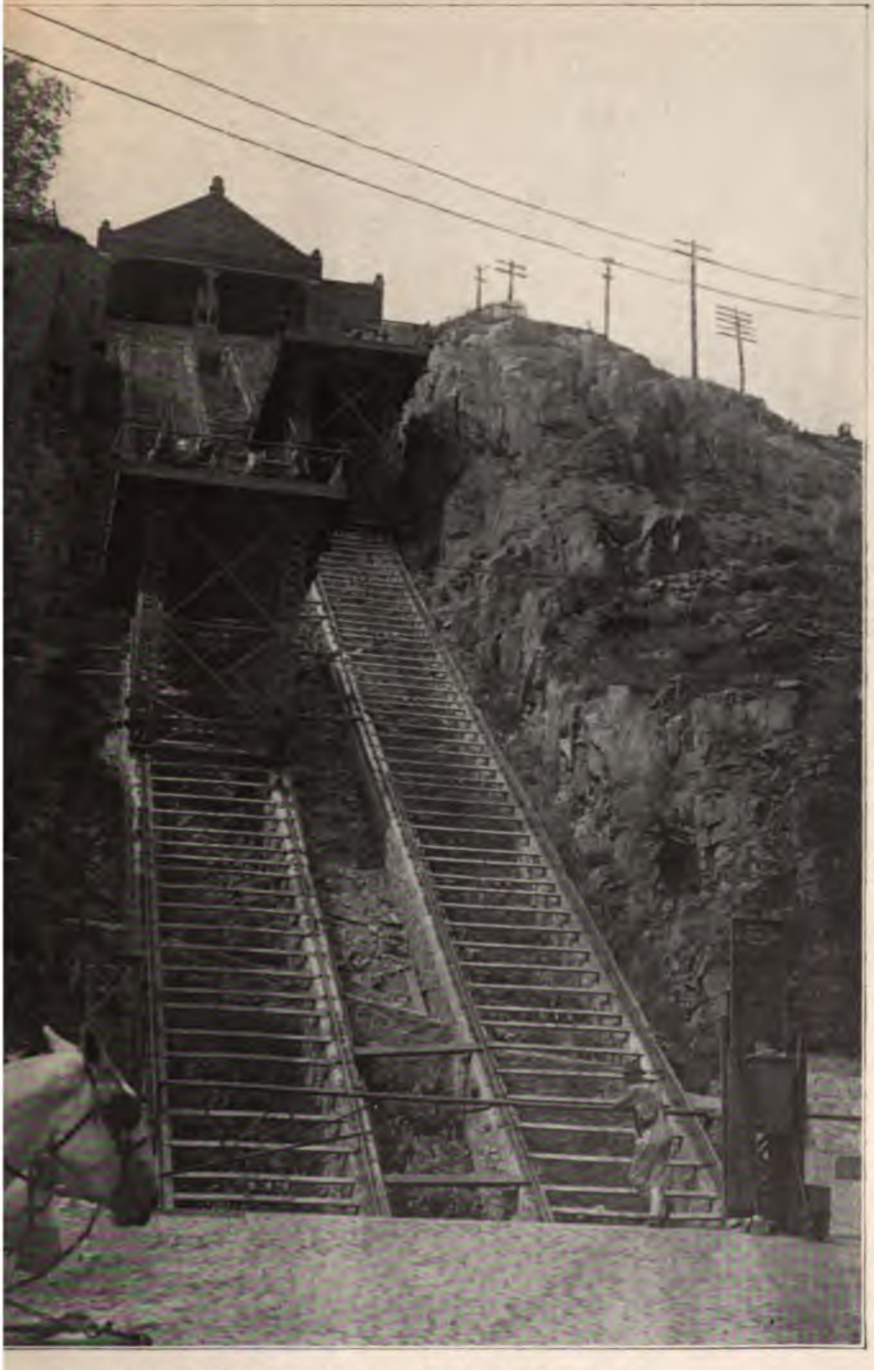
Solid rock will include all rock found in place in ledges and masses, or boulders measuring more than three cubic feet, and which can only be removed by blasting.

Prosecution of Earthwork. No general rule can be laid down for the exact method of carrying on an excavation and disposing of the excavated material. The operation in each case can only be determined by the requirements of the contract, character of the material, magnitude of the work, length of haul, etc.

Formation of Embankments. Where embankments are to be formed of less than two feet in height, all stumps, weeds, etc. should be removed from the space to be occupied by the embankment. For embankments exceeding two feet in height stumps need only be close cut. Weeds and brush, however, ought to be removed and if the surface is covered with grass sod, it is advisable to plow a furrow at the toe of the slope. Where a cutting passes into a fill all the vegetable matter should be removed from the surface before placing the fill. The site of the bank should be carefully examined and all deposits of soft, compressible matter removed. When a bank is to be made over a swamp or marsh, the site should be thoroughly drained, and if possible the fill should be started on hard bottom.

Perfect stability is the object aimed at, and all precautions necessary to this end should be taken. Embankments should be built in successive layers, banks two feet and under in layers from six inches to one foot, heavier banks in layers 2 and 3 feet thick. The horses and vehicles conveying the materials should be required to pass over the bank for the purpose of consolidating it, and care should be taken to have the layers dip towards the center. Embankments first built up in the center, and afterwards widened by dumping the earth over the sides, should not be allowed.

Embankments on Hillsides. When the axis of the road is laid out on the side slope of a hill, and the road is formed partly by excavating and partly by embanking, the usual and most simple method is to extend out the embankment gradually along the whole line of the excavation. This method is insecure; the excavated material if simply deposited on the natural slope is liable to slip, and no pains should be spared to give it a secure hold, particularly at the toe of the slope. The natural surface of the slope should be cut into steps as shown in Figs. 23 and 24. The dotted line A B



A PECULIAR EXAMPLE OF HIGHWAY CONSTRUCTION

ans of the truck elevators here shown, the roadway is carried over the steep bluffs of the Pal-
e Hudson at Hoboken, N. J.

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represents the natural surface of the ground. C E B the excavation, and A D C the embankment, resting on steps which have been cut between A and C. The best position for these steps is perpendicular to the axis of greatest pressure. If A D is inclined at the angle of repose of the material, the steps near A should be inclined in the

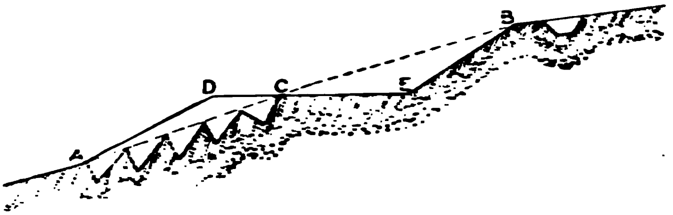

Fig. 23. Method of Construction on Hillsides

opposite direction to A D, and at an angle of nearly 90 degrees thereto, while the steps near C may be level. If stone is abundant, the toe of the slope may be further secured by a dry wall of stone.

On hillsides of great inclination the above method of construction will not be sufficiently secure; retaining walls of stone must be substituted for the side slopes of both the excavations and embankments. These walls may be made of stone laid dry, when stone

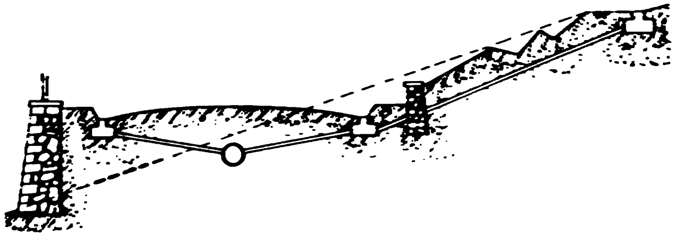

Fig. 24. Hillside Road with Retaining and Revetment Walls

can be procured in blocks of sufficient size to render this kind of construction of sufficient stability to resist the pressure of the earth. When the stones laid dry do not offer this security, they must be laid in mortar. The wall which forms the slope of the excavation should be carried up as high as the natural surface of the ground. Unless the material is such that the slope may be safely formed into steps or benches as shown in Fig. 23, the wall that sustains the embankment should be built up to the surface of the roadway, and a parapet wall or fence raised upon it, to protect pedestrians against accident. (See Fig. 24.)

For the formula for calculating the dimensions of retaining walls see instruction paper on Masonry Construction.

Roadways on Rock Slopes. On rock slopes when the inclination of the natural surface is not greater than one perpendicular to two base, the road may be constructed partly in excavation and partly in embankment in the usual manner, or by cutting the face of the slope into horizontal steps with vertical faces, and building up the embankment in the form of a solid stone wall in horizontal courses, laid either dry or in mortar. Care is required in proportioning the steps, as all attempts to lessen the quantity of excavation by increasing the number and diminishing the width of the steps require additional precautions against settlement in the built-up portion of the roadway.

When the rock slope has a greater inclination than 1 :2 the whole of the roadway should be in excavation.

In some localities roads have been constructed along the face of nearly perpendicular cliffs on timber frameworks consisting of horizontal beams, firmly fixed at one end by being let into holes drilled in the rock, the other end being supported by an inclined strut resting against the rock in a shoulder cut to receive it. There are also examples of similar platforms suspended instead of being supported.

Earth Roads. The term "earth road" is applied to roads where the surface consists of the native soil; this class of road is the most common and cheapest in first cost. At certain seasons of the year earth roads when properly cared for are second to none, but during the spring and wet seasons they are very deficient in the important requisite of hardness, and are almost impassable.

For the construction of new earth roads, all the principles previously discussed relating to alignment, grades, drainage, width, etc., should be carefully followed. The crown or transverse contour should be greater than in stone roads. Twelve inches at the center in 25 feet will be sufficient.

Drainage is especially important, because the material of the road is more susceptible to the action of water, and more easily destroyed by it than are the materials used in the construction of the better class of roads. When water is allowed to stand upon the road, the earth is softened, the wagon wheels penetrate it and the horses' feet mix and kneed it until it becomes impassable mud. The action of frost is also apt to be more disastrous upon the more permeable surface of the earth road, having the effect of swelling and heaving the roadway and throwing its surface out of shape. It may in fact be said that the whole problem of the improvement and maintenance of ordinary country roads is one of drainage.

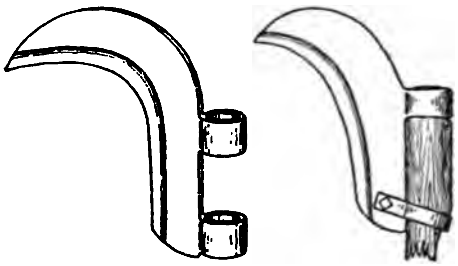

Fig. 25. Bush Hooks.

In the preparation of the wheelway all stumps, brush, vegetable matter, rocks and boulders should be removed from the surface and the resulting holes filled in with clean earth. The roadbed having been brought to the required grade and crown should be thoroughly rolled, all inequalities appearing during the rolling should be filled up and re-rolled.

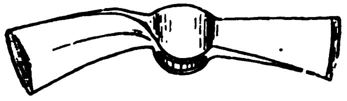

Fig. 26. Axe Mattock.

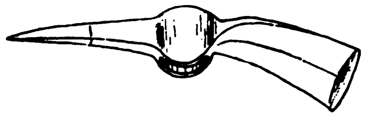

Fig. 27. Bush Mattock.

Care of Earth Roads. If the surface of the roadway is properly formed and kept smooth, the water will be shed into the side ditches and do comparatively little harm; but if it remains upon the surface, it will be absorbed and convert the road into mud. All ruts and depressions should be filled up as soon as they appear. Repairs should be attended to particularly in the spring. At this season a judicious use of a road machine and rollers will make a smooth road. In summer when the surface gets roughed up it can be improved by running a harrow over it; if the surface is a little muddy this treatment will hasten the drying.

During the fall the surface should be repaired, with special reference to putting it in shape to withstand the ravages of winter. Saucer-like depressions and ruts should be filled up with clean earth similar to that of the roadbed and tamped into place.

The side ditches should be examined in the fall to see that they are free from dead weeds and grass, and late in winter they should be examined again to see that they are not clogged. The mouths of culverts should be cleaned of rubbish and the outlet of tile drains opened. Attention to the side ditches will prevent overflow, and washing of the roadway, and will also prevent the formation of ponds at the roadside and the consequent saturation of the roadbed.

Holes and ruts should not be filled with stone, bricks, gravel or other material harder than the earth of the roadway as the hard material will not wear uniform with the rest of the road, but produce bumps and ridges, and usually result in making two holes, each larger than the original one. It is bad practice to cut a gutter from a hole to drain it to the side of the road. Filling is the proper course, whether the hole is dry or contains mud.

In the maintenance of clay roads neither sods nor turf should be used to fill holes or ruts; for, though at first deceptively tough, they soon decay and form the softest mud. Neither should the ruts be filled with field stones; they will not wear uniformly with the rest of the road, but will produce hard ridges.

Trees and close hedges should not be allowed within 200 feet of a clay road. It requires all the sun and wind possible to keep its surface in a dry and hard condition.

Sand Roads. The aim in the improvement of sand roads is to have the wheelway as narrow and well defined as possible, so as to have all the vehicles run in the same track. An abundant growth of vegetation should be encouraged on each side of the wheelway, for by this means the shearing of the sand is, in a great measure, avoided. Ditching beyond a slight depth to carry away the rain water is not desirable, for it tends to hasten the drying of the sand, which is to be avoided. Where possible the roads should be over-

EXAMPLE OF A BAD ROAD THAT COULD BE MADE AN EXCELLENT ONE WITH BUT LITTLE LABOR AND EXPENSE

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hung with trees, the leaves and twigs of which catching on the wheelway will serve still further to diminish the effect of the wheels in moving the sands about. If clay can be obtained, a coating 6 inches thick will be found a most effective and economical improvement. A coating of 4 inches of loose straw will, after a few days' travel, grind into the sand and become as hard and as firm as a dry clay road.

The maintaining of smooth surfaces on all classes of earth roads will be greatly assisted and cheapened by the frequent use of a roller (either steam or horse) and any one of the various forms of road grading and scraping machines. In repairing an earth road the plough should not be used. It breaks up the surface which has been compacted by time and travel.

TOOLS FOR GRADING.

Picks are made of various styles, according to the class of material in which they are to be used. Fig. 28 shows the form usually employed in street work. Fig. 29 shows the form generally used for clay or gravel excavation.

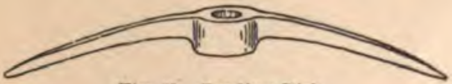
Fig. 28. Grading Pick.

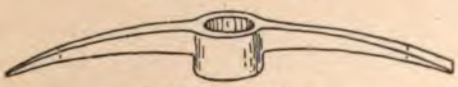
Fig. 29. Clay Pick.

The eye of the pick is generally formed of wrought iron, pointed with steel. The weight of picks ranges from 4 to 9 lb.

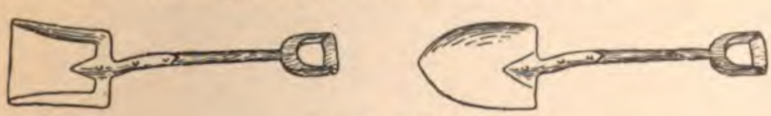
Fig. 30. Shovels.

Shovels are made in two forms, square and round pointed, usually of pressed steel.

Ploughs are extensively employed in grading, special forms being manufactured for the purpose. They are known as "grading ploughs," "road ploughs," "township ploughs," etc. They vary in form according to the kind of work they are intended for, viz.: loosening earth, gravel, hardpan, and some of the softer rocks.

These ploughs are made of great strength, selected white oak, rock elm, wrought steel and iron being generally used in their construction. The cost of operating ploughs ranges from 2 to 5 cents per cubic yard, depending upon the compactness of the soil. The quantity of material loosened will vary from 2 to 5 cubic yards per hour.

Fig. 31 shows the form usually adopted for loosening earth. This plough does not turn the soil, but cuts a furrow about 10 inches wide and of a depth adjustable up to 11 inches.

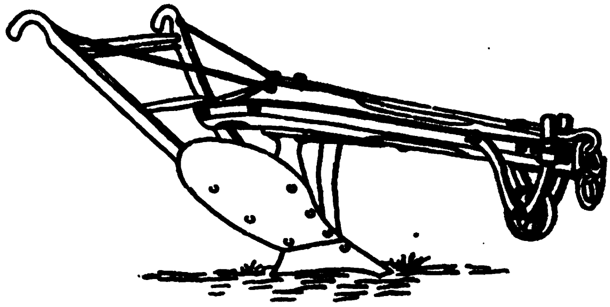

Fig. 31. Grading Plow.

In light soil the ploughs are operated by two or four horses; in heavy soils as many as eight are employed. Grading ploughs vary in weight from 100 to 325 lb.

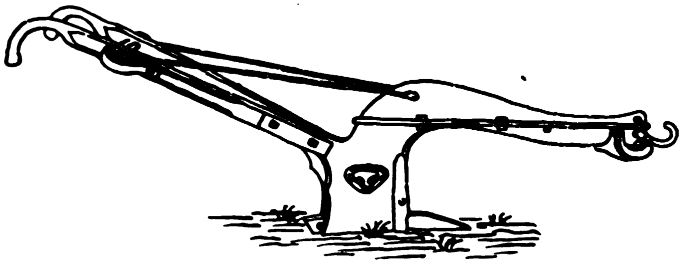

Fig. 32. Hardpan Plow.

Fig. 32 illustrates a plough specially designed for tearing up macadam, gravel, or similar material. The point is a straight bar of cast steel drawn down to a point, and can be easily repaired.

Scrapers are generally used to move the material loosened by ploughing; they are made of either iron or steel, and in a variety of form, and are known by various names, as "drag," "buck," "pole," and "wheeled". The drag scrapers are usually employed on short hauls, the wheeled on long hauls. Fig. 33 illustrates the usual form of drag scrapers.

Drag scrapers are made in three sizes. The smallest, for one horse, has a capacity of 3 cubic feet; the others, for two horses, have a capacity of 5 to 7½ cubic feet. The smallest weighs about 90 lb., and the larger ones from 94 to 102 lb.

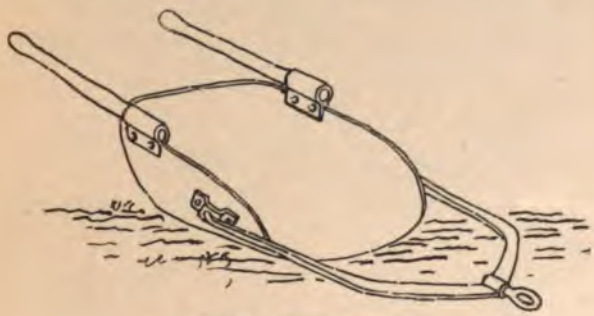

Fig. 33. Drag Scraper.

Buck scrapers are made in two sizes—two-horses, carrying 7½ cubic feet; four-horses, 12 cubic feet.

Pole scraper, Fig. 34, is designed for use in making and leveling earth roads and for cutting and cleaning ditches; it is also well adapted for moving earth short distances at a minimum cost.

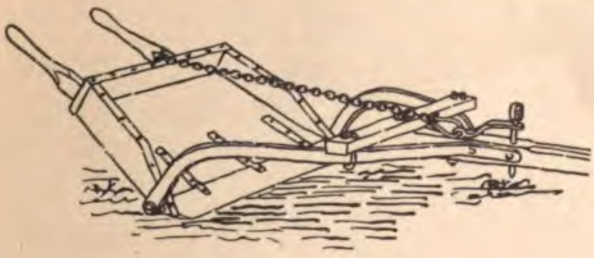

Fig. 34. Pole Scraper.

Wheeled scrapers consist of a metal box, usually steel, mounted on wheels, and furnished with levers for raising, lowering, and dumping. They are operated in the same manner as drag scrapers, except that all the movements are made by means of the levers, and without stopping the team. By their use the excessive resistance to traction of the drag scraper is avoided. Various sizes are made, ranging in capacity from 10 to 17 cubic feet. In weight they range from 350 to 700 lb.

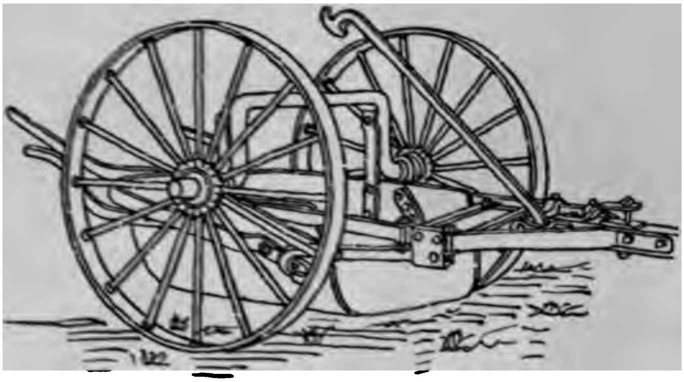

Fig. 35. Wheeled Scraper.

Wheelbarrows. The wheelbarrow shown in Fig. 36 is constructed of wood and is the most commonly employed for earthwork. Its capacity ranges from 2 to 2½ cubic feet. Weight about 50 lb.

The barrow, Fig. 37, has a pressed-steel tray, oak frame, and steel wheel, and will be found more durable in the maintenance department than the all wood barrow. Capacity from 3½ to 5 cubic feet, depending on size of tray.

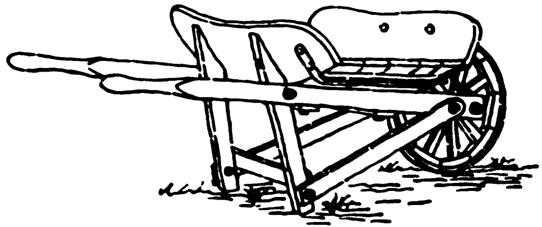

Fig. 36. Wooden Barrow.

The barrow, Fig. 38, is constructed with tubular iron frames and steel tray, and is adaptable to the heaviest work, such as moving heavy broken stone, etc., or it may be employed with advantage in the cleaning department. Capacity from 3 to 4 cubic feet. Weight from 70 to 82 lb.

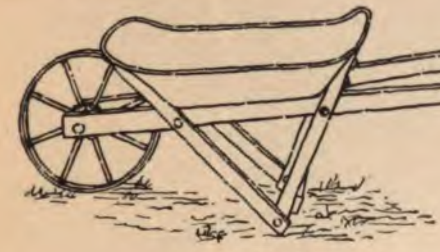

Fig. 37. Steel Tray Barrow.

The maximun distance to which earth can be moved economically in barrows is about 200 feet. The wheeling should be performed upon planks, whose steepest inclination should not exceed 1 in 12. The force required to move a barrow on a plank is about $\frac{1}{25}$ part of the weight; on hard dry earth, about $\frac{1}{14}$ part of the weight.

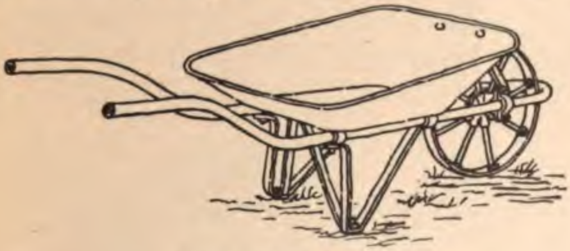

Fig. 38. Metal Barrow.

The time occupied in loading a barrow will vary with the character of the material and the proportion of wheelers to shovellers. Approximately, a shoveller takes about as long to fill a barrow with earth as a wheeler takes to wheel a full barrow a distance of about 100 or 120 feet on a horizontal plank and return with the empty barrow.

Carts. The cart usually employed for hauling earth, etc., is shown in Fig. 39. The average capacity is 22 cubic feet, and the average weight is 800 lb. These carts are usually furnished with broad tires, and the body is so balanced that the load is evenly divided about the axle.

The time required to load a cart varies with the material. One shoveller will require about as follows: Clay, seven minutes; loam, six minutes; sand, five minutes.

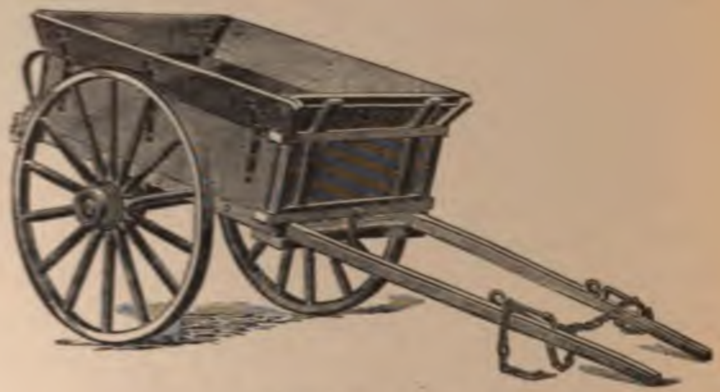

Fig. 39. Earth Wagon.

Dump Cars. These cars are made to dump in several different ways, viz., single or double side, single or double end, and rotary or universal dumpers.

Dump cars may be operated singly or in trains, as the magnitude of the work may demand. They may be moved by horses or small locomotives. They are made in various sizes, depending upon the gauge of the track on which they are run. A common gauge is

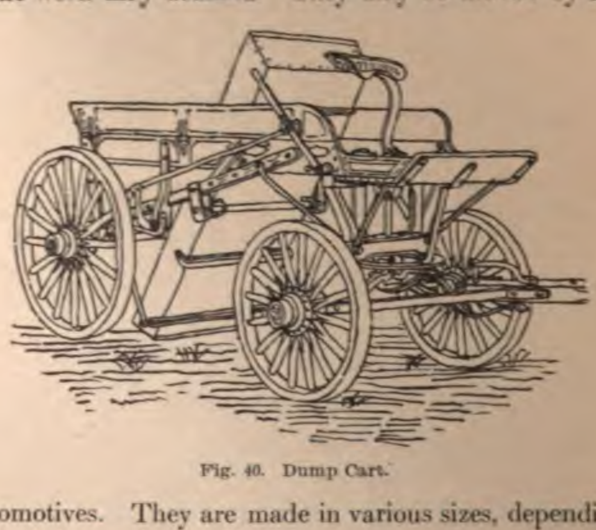

Fig. 40. Dump Cart.

20 inches, but it varies from that up to the standard railroad gauge of 56½ inches.

Dump Wagons. (Fig. 40.) The use of these wagons for moving excavated earth, etc., and for transporting materials such as sand, gravel, etc., materially shortens the time required for unloading the ordinary form of contractor's wagon; having no reach or pole connecting the rear axle with the center bearing of the front axle, they may be cramped short and the load deposited just where required. They are operated by the driver, and the capacity ranges from 35 to 45 cubic feet.

Mechanical Graders are used extensively in the making and maintaining of earth roads. They excavate and move earth more expeditiously and economically than can be done by hand; they are called by various names, such as "road machines," "graders," "road hones," etc. Their general form is shown in Fig. 41.

Briefly described, they consist of a large blade made entirely of steel or of iron, or wood shod with steel, which is so arranged by mechanism attached to the frame from which it is suspended that it can be adjusted and fixed in any direction by the operator. In their action they combine the work of excavating and transporting the earth. They have been chiefly employed in the forming and maintenance of earth roads, but may be also advantageously used in preparing the subgrade surface of roads for the reception of broken stone or other improved covering.

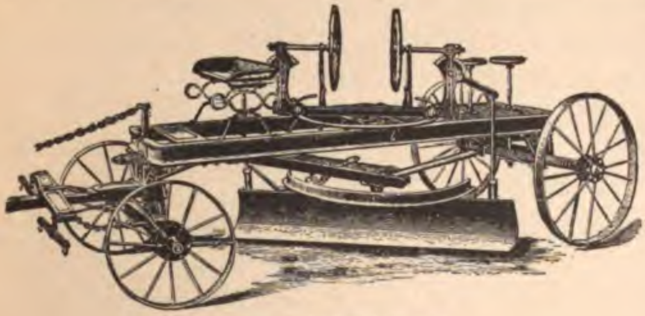

Fig. 41. Mechanical Grader.

A large variety of such machines are on the market. The "New Era" grader excavates the material from side ditches, and automatically loads the material into carts or wagons. Briefly described, the machine consists of a plough which loosens and raises the earth, depositing it upon a transverse carrying-belt, which conveys it from excavation to embankment. This carrier is built in four sections, bolted together, so it can be used to deliver earth at 14, 17, 19, or 22 feet from the plough. The carrier belt is of heavy 3-ply rubber 3 feet wide.

The plough and carrier are supported by a strong trussed framework resting on heavy steel axles and broad wheels. The large rear wheels are ratcheted upon the axle, and connected with strong gearing which propels the carrying-belt at right angles to the direction in which the machine is moving.

The wheels and trusses are low and broad, occupying a space 8 feet wide and 14 feet long, exclusive of the side carrier. This enables it to work on hillsides where any wheeled implements can be used. Notwithstanding its large size it is so flexible that it may be turned around on a 16-foot embankment. Pilot wheels and levers enable the operator to raise or lower the plough or carrier at pleasure.

As a motive power 12 horses—8 driven in front, 4 abreast, and 4 in the rear on a push cart—are usually employed.

When the teams are started, the operator lowers the plough and throws the belting in gear, and as the plough raises and turns the earth to the side the carrier receives and delivers it at the distance for which the carrier is adjusted, forming either excavation or embankment.

When it becomes necessary to deliver the excavated earth beyond the reach of the carrier (22 feet or 7½ feet above the plough), the earth is loaded upon wagons, then conveyed to any distance. Arranging the carrier at 19 feet, wagons are driven under the carrier and loaded with 1½ to 1[illegible] yards of earth in from 20 to 30 seconds. When one wagon turns out with its load, another drives under the carrier, and the machine thus loads 600 to 800 wagons per day. It is claimed that with six teams and three men it is capable of excavating and placing in embankment from 1000 to 1500 cubic yards of earth in ten hours, or of loading from 600 to 800

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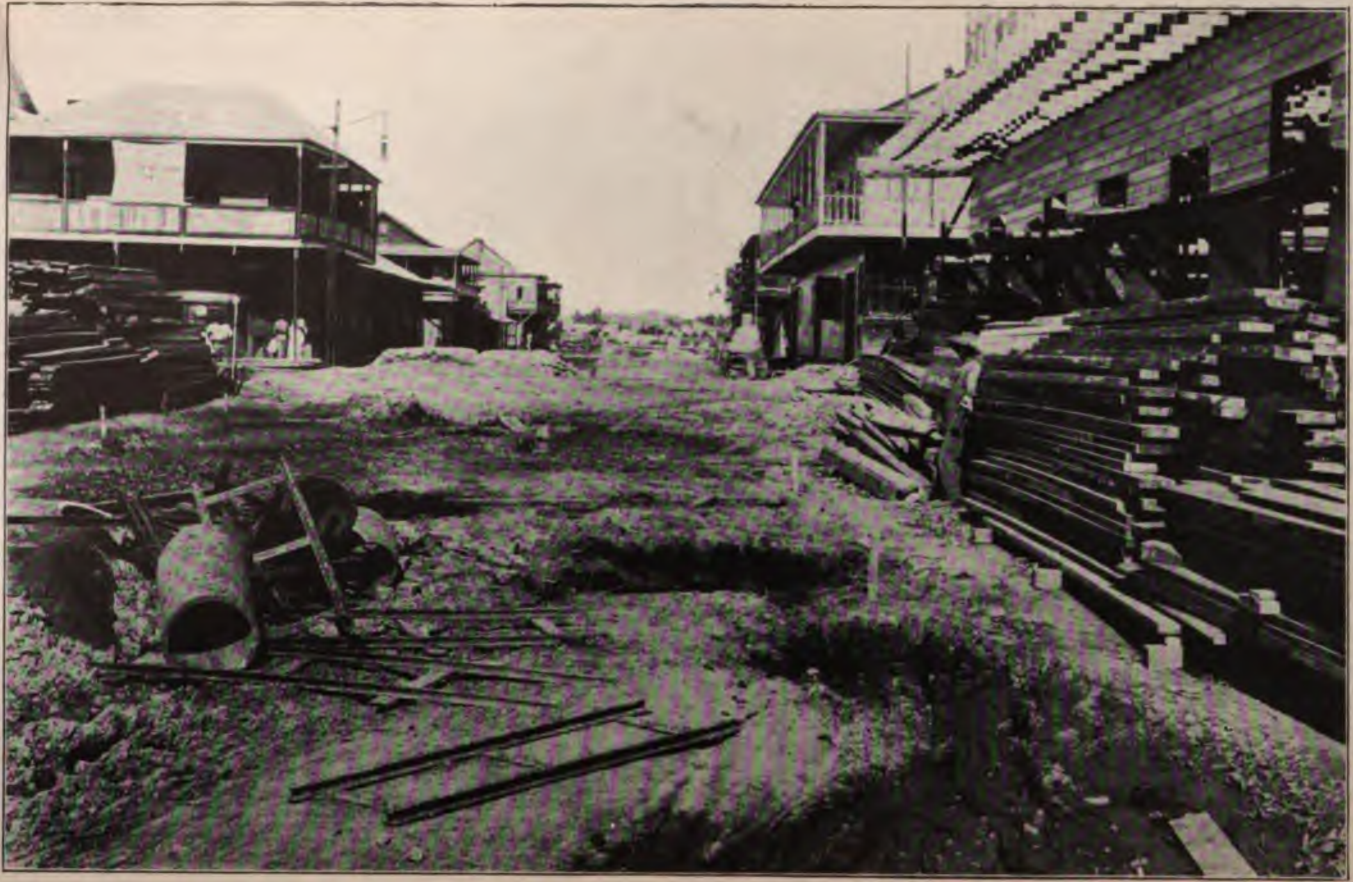

IMPROVEMENT OF THE CITY OF COLON, PANAMA

Fourth Street before paving. For view of same street after paving, see page 331.

wagons in the same time, and that the cost of this handling is from 1½ to 2½ cents per cubic yard.

Points to be Considered in Selecting a Road Machine. In the selection of a road machine the following points should be carefully considered:

(1) Thoroughness and simplicity of its mechanical construction.
(2) Material and workmanship used in its construction.
(3) Ease of operation.
(4) Lightness of draft
(5) Adaptability for doing general road-work, ditching, etc.
(6) Safety to the operator.

Care of Road Machines. The road machine when not in use should be stored in a dry house and thoroughly cleaned, its blade brushed clean from all accumulations of mud, wiped thoroughly dry, and well covered with grease or crude oil. The axles, journals, and wearing parts should be kept well oiled when in use, and an extra blade should be kept on hand to avoid stopping the machine while the dulled one is being sharpened.

Surface Graders. The surface grader, Fig. 42, is used for removing earth previously loosened by a plough. It is operated by one horse. The load may be retained and carried a considerable distance, or it may be spread gradually as the operator desires. It is also employed to level off and trim the surface after scrapers.

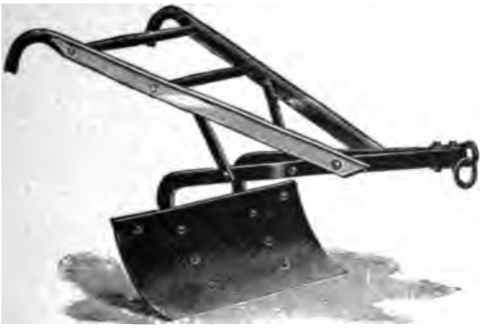

Fig. 42. Surface Grader.

The blade is of steel, ¼-inch thick, 15 inches wide, and 30 inches long. The beam and other parts are of oak and iron Weight about 60 lb.

The road leveller, Fig. 43, is used for trimming and smoothing the surface of earth roads. It is largely employed in the Spring when the frost leaves the ground.

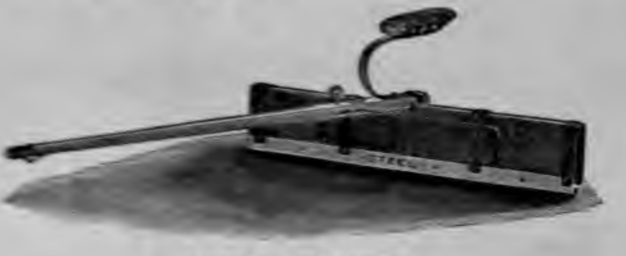

Fig. 43. Road Leveller.

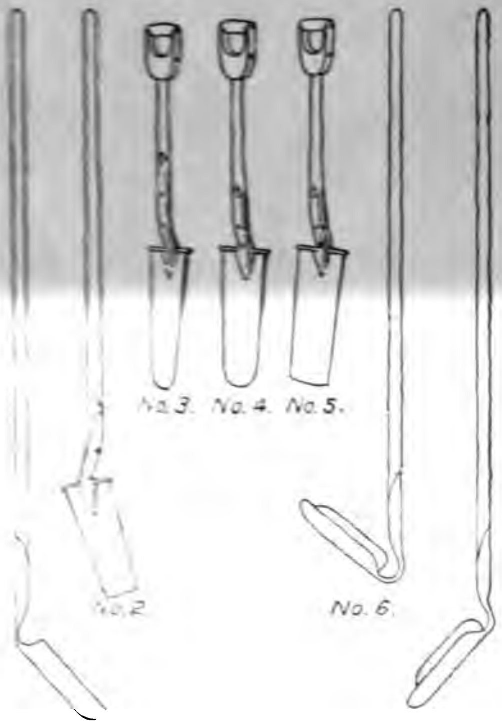

Fig. 44. Draining Tools.

The blade is of steel, ¼-inch thick by 4 inches by 72 inches, and is provided with a seat for the driver. It is operated by a team of horses. Weight about 150 lb.

Draining-tools. The tools employed for digging the ditches and shaping the bottom to fit the drain tiles are shown in Fig. 44. They are convenient to use, and expedite the work by avoiding unnecessary excavation.

The tools are used as follows: Nos. 3, 4 and 5 are used for digging the ditches; Nos. 6 and 7 for cleaning and rounding the bottom of the ditch for round tile. No. 2 is used for shoveling out loose earth and levelling the bottom of the ditch; No. 1 is used for the same purpose when the ditch is intended for "sole" tile.

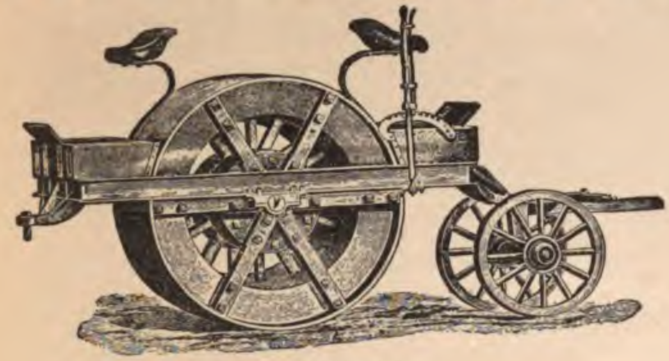

Fig. 45. Reversible Roller.

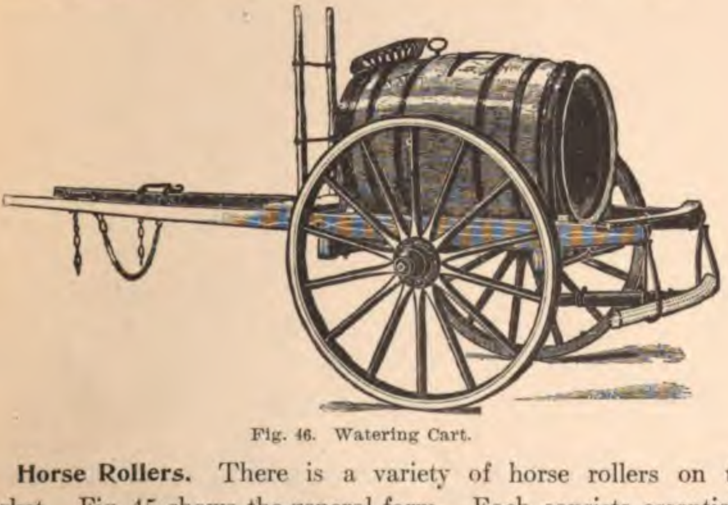

Fig. 46. Watering Cart.

Horse Rollers. There is a variety of horse rollers on the market. Fig. 45 shows the general form. Each consists essentially of a hollow cast-iron cylinder 4 to 5 feet long, 5 to 6 feet in diameter, and weighing from 3 to 6 tons. Some forms are provided with boxes in which stone or iron may be placed to increase the weight, and some have closed ends and may be filled with water or sand.

Sprinkling-carts. Fig. 46 shows a convenient form of sprinkling cart for suburban streets and country roads. Capacity about 150 gallons.

ROAD COVERINGS.

Road coverings consist of some foreign material as gravel, broken stone, clay, etc., placed on the surface of the earth road. The object of this covering, whatever its nature, is (1) to protect the natural soil from the effect of weather and travel, and (2) to furnish a smooth surface on which the resistance to traction will be reduced to the least possible amount, and over which vehicles may pass with safety and expedition at all seasons of the year. Where an artificial covering is employed, the wheel loads coming upon its surface are distributed over a greater area of the roadbed than if the loads come directly upon the earth itself. The loads are not sustained by the covering as a rigid structure, but are transferred through it to the roadbed, which must support both the weight of the covering and the load coming upon it.

Gravel Roads. Gravel is an accumulation of small more or less rounded stones which usually vary from the size of a small pea to a walnut. It is often intermixed with other substances, such as sand, clay, loam, etc., from each of which it derives a distinctive name. In selecting gravel for road purposes the chief quality to be sought for is the property of binding.

Gravel in general is unserviceable for roadmaking. This is due mainly to the fact that the surface of the pebbles is smooth, so that they will not bind together in the manner of broken stone. There is also an absence of dust or other material to serve as a binder, and even if such binding material is furnished it is difficult to effectively hold the rounded and polished surface of the pebbles together.

In certain deposits of gravel, particularly where the pebbly matter is to a greater or less extent composed of limestone, a considerable amount of iron oxide has been gathered in the mass.

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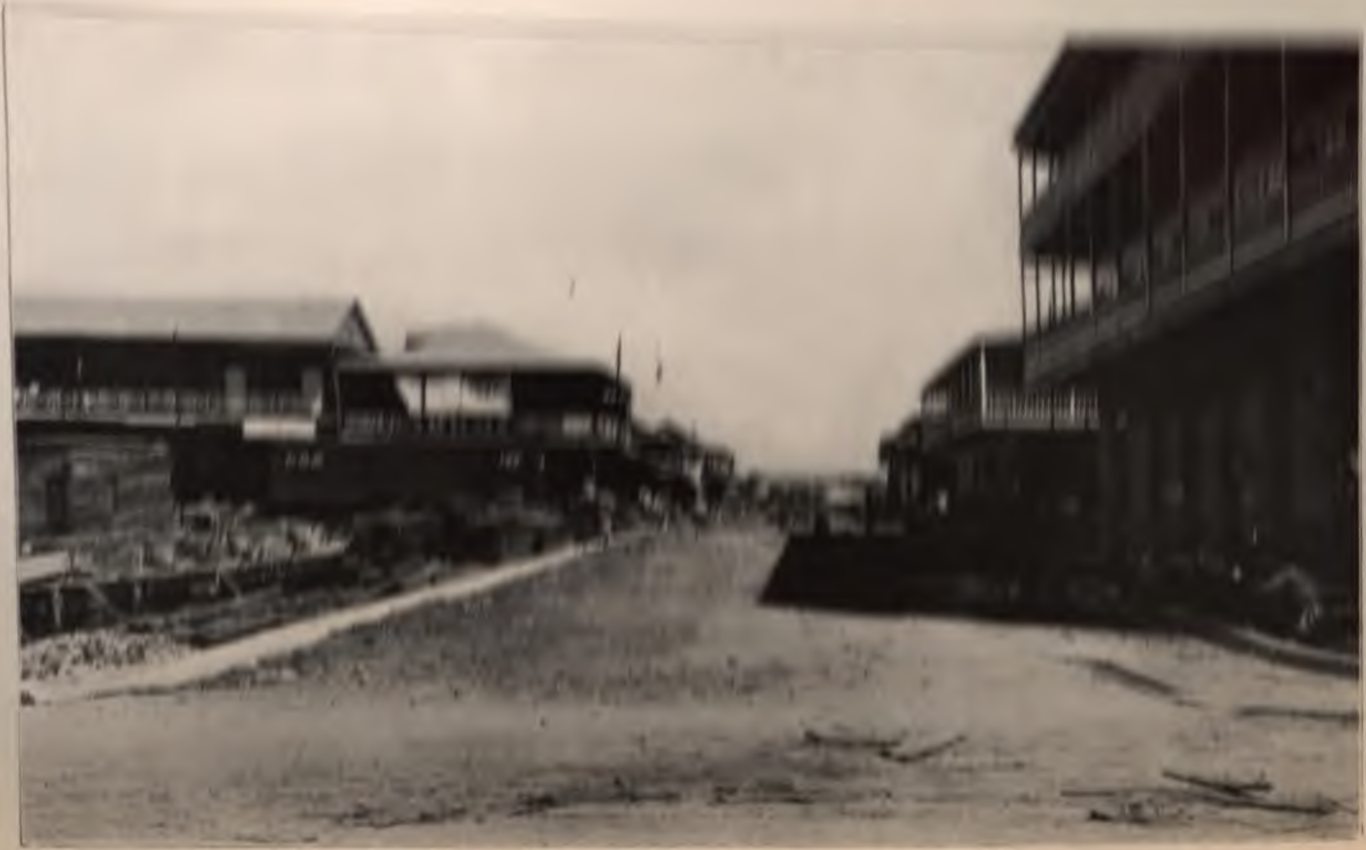

IMPROVEMENT OF THE CITY OF COLON, PANAMA

Fourth Street after paving. For view of same street before improvement, see page 327.

This effect is due to the tendency of water which contains iron to lay down that substance and to take lime in its place when the opportunity for so doing occurs. Such gravels are termed ferruginous. They are commonly found in a somewhat cemented state, and when broken up and placed upon roads they again cement, even more firmly than in the original state, often forming a roadway of very good quality.

When no gravel but that found in rivers or on the seashore can be obtained, one-half of the stone should be broken and mixed with the other half; to the stone so mixed a small quantity of clay or loam, about one-eighth of the bulk of the gravel, must be added: an excess is injurious. Sand is unsuitable. It prevents packing in proportion to the amount added.

Preparing the Gravel. Pit gravel usually contains too much earth, and should be screened before being used. Two sieves should be provided, one with meshes of one and one-half inches, so that all pebbles above that size may be rejected, the other with meshes of three quarters of an inch, and the material which passes through it should be thrown away. The expense of screening will be more than repaid by the superior condition of the road formed by the cleaned material, and by the diminution of labor in keeping it in order. The pebbles larger than one and a half inches may be broken to that size and mixed with clean material.

Laying the Gravel. On the roadbed properly prepared a layer of the prepared gravel four inches thick is uniformly spread over the whole width, then compacted with a roller weighing not less than two tons, and having a length of not less than thirty inches. The rolling must be continued until the pebbles cease to rise or creep in front of the roller. The surface must be moistened by sprinkling in advance of the roller, but too much water must not be used. Successive layers follow, each being treated in the above described manner until the requisite depth and form has been attained.

The gravel in the bottom layer must be no larger than that in the top layer; it must be uniformly mixed, large and small together, for if not, the vibration of the traffic and the action of frost will cause the larger pebbles to rise to the surface and the smaller ones to descend, and the road will never be smooth or firm.

[illegible] imbedded in a paste [illegible]. It is for this reason, among others, [illegible] destruction.

[illegible] depends in a large measure upon [illegible]. During rainy periods a gravel road be[illegible] while in very dry weather the clay will con[illegible] the pebbles, and giving a loose [illegible] are obtained in moderately [illegible] a gravel road offers several [illegible] of the drainage, etc., [illegible] etc.

[illegible] above described will [illegible] daily attention is required [illegible] should be kept at hand to draw [illegible] and for filling any depres[illegible].

[illegible] a small quantity of gravel [illegible] through. Two [illegible] than a larger amount. [illegible] it does not all pack, and if, [illegible] gravel will be found: [illegible]

[illegible] gravel [illegible] fast as [illegible]

[illegible] earth [illegible] This class [illegible] road [illegible] type into [illegible]

[illegible] that [illegible] disregarded this foundation, contending that the subsoil, however bad, would carry any weight if made dry by drainage and kept dry by an impervious covering. The names of both have ever since been associated with the class of road which each favored, as well as with roads on which all their precepts have been disregarded.

Quality of Stones. The materials used for broken-stone pavements must of necessity vary very much according to the locality. Owing to the cost of haulage, local stone must generally be used, especially if the traffic be only moderate. If, however, the traffic is heavy, it will sometimes be found better and more economical to obtain a superior material, even at a higher cost, than the local stone; and in cases where the traffic is very great, the best material that can be obtained is the most economical.

The qualities required in a good road stone are hardness and and toughness and ability to resist the disintegrating action of the weather. These qualities are seldom found together in the same stone. Igneous and siliceous rocks, although frequently hard and tough, do not consolidate so well nor so quick as limestone, owing to the sandy detritus formed by the two first having no cohesion, whilst the limestone has a detritus which acts like mortar in binding the stones together.

A stone of good binding nature will frequently wear much better than one without, although it is not so hard. A limestone road well made and of good cross-section will be more impervious than any other, owing to this cause, and will not disintegrate so soon in dry weather, owing partly to this and partly to the well-known quality which all limestone has of absorbing moisture from the atmosphere. Mere hardness without toughness is not of much use, as a stone may be very hard but so brittle as to be crushed to powder under a heavy load, while a stone not so hard but having a greater degree of toughness will be uninjured.

By a stone of good binding quality is meant one that, when moistened by water and subjected to the pressure of loaded wheels or rollers, will bind or cement together. This quality is possessed to a greater or less extent by nearly all rocks when in a state of disintegration. The binding is caused by the action of water upon the chemical constituents of the stone contained in the detritus produced g the stone, and by the friction of the fragments on each while being compacted; its strength varies with the different ies of rock, but it exists in some measure with them all, being test with limestone and least with gneiss.

The essential condition of the stone to produce this binding fect is that it be sound. No decayed stone retains the property of binding, though in some few cases, where the material contains iron xides, it may, by the cementing property of the oxide, undergo a ertain binding.

A stone for a road surface should be as little absorptive of isture as possible in order that it may not suffer injury from the ion of frost. Many limestones are objectionable on this account.

The stone used should be uniform in quality, otherwise it will wear unevenly, and depressions will appear where the softer material s been used.

As the under parts of the road covering are not subject to the ear of traffic, and have only the weight of loads to sustain, it is t necessary that the stone of the lower layer be so hard or so igh as the stone for the surface, hence it is frequently possible by ng an inferior stone for that portion of the work, to greatly reduce the cost of construction.

Size of Stones. The stone should be broken into fragments as nearly cubical as possible. The size of the cubes will depend upon the character of the rock. If it be granite or trap, they should not exceed 1½ inches in their greatest dimensions; if limestone, they should not exceed 2 inches.

The smaller the stones the less the percentage of voids. Small stones compact sooner, require less binding, and make a smoother surface than large ones, but the size of the stone for any particular section of a road must be determined to a certain extent by the amount of traffic which it will have to bear and the character of the rock used.

It is not necessary nor is it advisable that the stone should be all of the same size; they may be of all sizes under the maximum. In this condition the smaller stones fill the voids between the larger and less binding is required.

Thickness of the Broken Stone. The offices of the broken stone are to endure friction and to shed water; its thickness must be regulated by the quality of the stone, the amount of traffic, and nature of the natural soil bed. Under heavy traffic it is advisable to make the thickness greater than for light traffic, in order to provide for wear and lessen the cost of renewals.

When the roadbed is firm, well drained, and not likely to be affected by ground water, it will always afford a firm foundation for the broken stone, the thickness of which may be made the minimum for good construction. This thickness is four inches. When this thickness is employed the stone must be of exceptionally fine quality and the road must be maintained by the "continuous" method. With heavy traffic the thickness should be increased over the minimum a certain amount, say 2 inches, to provide for wear. Where the foundation is unstable and there is a tendency on the part of the loads to break through the covering, the thickness of the stone must be made the maximum, which is 12 inches. In such a case it may be advisable to employ a Telford foundation. Where the covering exceeds six inches in thickness, the excess may be composed of gravel, sand or ledge stone, the choice depending entirely on the cost, for all are equally effective.

Foundation. The preparation of the natural soil over which the road is to be constructed, to enable it to sustain the superstructure and the weights brought upon it, requires the observance of certain precautions the neglect of which will sooner or later result in the deterioration or possible destruction of the road covering. These precautions vary with the character of the soil.

Soils of a siliceous and calcareous nature do not present any great difficulty, as their porous nature generally affords good natural drainage which secures a dry foundation. Their surface, however, requires to be compacted; this is effected by rolling.

The rolling should be carried out in dry weather, and any depressions caused by the passage of the roller should be filled with the same class of material as the surrounding soil. The rolling must be repeated until a uniform and solid bed is obtained.

The argillaceous and allied soils, owing to their retentive nature, are very unstable under the action of water and frost, and in their natural condition afford a poor foundation. The preparation of such soil is effected by drainage and by the application of a layer of suitable material to entirely separate the surface from the road material. This material may be sand, furnace ashes, or other material of a similar nature, spread in a layer from 3 to 6 inches thick over the surface of the natural soil.

When the road is formed in rock cuttings it is advisable to spread a layer of sand or other material of light nature, so as to fill up the irregularities of the surface as well as to form a cushion for the road material to rest on.

Spreading the Stone. The stone should be hauled upon the roadbed in broad-tire two-wheeled carts and dumped in heaps and be spread evenly with a rake in a layer which should be of a depth of 4½ inches.

Watering. Wetting the stone expedites the consolidation, decreases crushing under the roller, and assists the filling of the voids with the binder. It should be applied by a sprinkler and should not be thrown on in quantity or from the plain nozzle of a hose.

Excessive watering, especially in the earlier stages, tends to soften the foundation, and care should be exercised in its application.

Binder. [illegible] filling [illegible] material [illegible] and [illegible] sufficient. [illegible] may be [illegible] binder [illegible] not be [illegible] in small [illegible] with the [illegible]

[illegible] importance, the [illegible] material [illegible] initial cost of the [illegible] greater when a good binding material is used.

Stone consolidated with improper binding material may present a good appearance immediately after being rolled and be otherwise an apparently good piece of work, still in damp weather a considerable amount of "licking up" by the wheels of the vehicles will take place, which reduces the strength of the coating and causes the surface to wear unequally.

By the application of an immoderate quantity of binding of any description the stone coating will become unsound or rotten in condition, and if the binding be of an argillaceous nature, it will expand during frost, owing to its absorbent properties, and cause the displacement of the stones. The surface will become sticky, which seriously affects the tractive power of horses, while the road itself will suffer by the irregular deterioration of the surface.

The use of such material as mentioned for binding enables rolling to be accomplished in much less time than when proper binding is used, and the cost of consolidating the stone may be reduced by 25 per cent; but, on the other hand, the stone coating which will probably contain under these circumstances from 30 to 40 per cent of soft and soluble matter, and possibly present a smooth surface immediately after being rolled, will quickly become "cupped" by the wheel traffic a bumpy surface being the result. This is caused by the irregular wear, while the lasting qualities or "life" of the coating will be shortened, giving unsatisfactory results to those traveling over the road, and the work of renewing the surface of the road in this manner may prove a failure on economical grounds. There can be no doubt, and it is now being more generally recognized, that sand as a material for binding in connection with rolling operations, when applied in a limited but sufficient quantity, promotes the durability of the stone coating, while the general results are equally satisfactory; a firm, compact, and smooth surface is obtained, and the subsequent maintenance of the road is minimized.

A great amount of rolling is necessary when sand is employed as a binding material, but economy is promoted, and the results are more satisfactory when sand is used than by the use of the material which gives to the stone an appearance only of having been properly consolidated. If clean sand be used in combination with the screenings from the crusher a very satisfactory surface will be obtained.

use of motor vehicles equipped with pneumatic tires be- general, it is possible that some other description of binding al will be necessary. The pumping action of suction created pneumatic tires, especially when propelled at a high speed, causes onsiderable movement of the fine particles of the binding material, h on being displaced will convert the covering into a mass of . This objection can probably be overcome by watering.

ompacting the Broken Stone. Three methods of compacting ne broken stone are practiced: (1) by the traffic passing over the road;) by rollers drawn by horses; (3) by rollers propelled by steam.

The first method is both defective and objectionable. (1) It is ructive to the horses and vehicles using the road. (2) It is waste- of material; about one-third of the stone is worn away in the operation. (3) Dung and dust are ground up with the stone, and the road is more readily affected by wet and frost.

Steam-rollers were first successfully introduced in France in 1860, e which time they have been almost universally adopted on account the superiority and economy of the work done. Their use shortens time required for construction or repair, and effects an indirect ing by the reduced wear and tear of horses and vehicles. They are made in different weights ranging from 3 to 30 tons. For the compacting of broken stone roads the weights in favor are from ten to fifteen tons; the heavier weights are considered unwieldy and their use is liable to cause damage to the underground structures that may be in the roadway.

The advantage of steam rolling may be summed up as follows:

(1) They shorten the time of construction.

(2) A saving of road material, (a) because there are no loose stones to be kicked about and worn; (b) because there is no abrasion of the stone, only one surface of the stone being exposed to wear; (c) because a thinner coating of stone can be employed; (d) because no ruts can be formed in which water can lie to rot the stone.

(3) Steam-rolled roads are easier to travel on account of their even surface and superior hardness and they have a better appearance.

(4) The roads can be repaired at any season of the year.

(5) Saving both in materials and manual labor.

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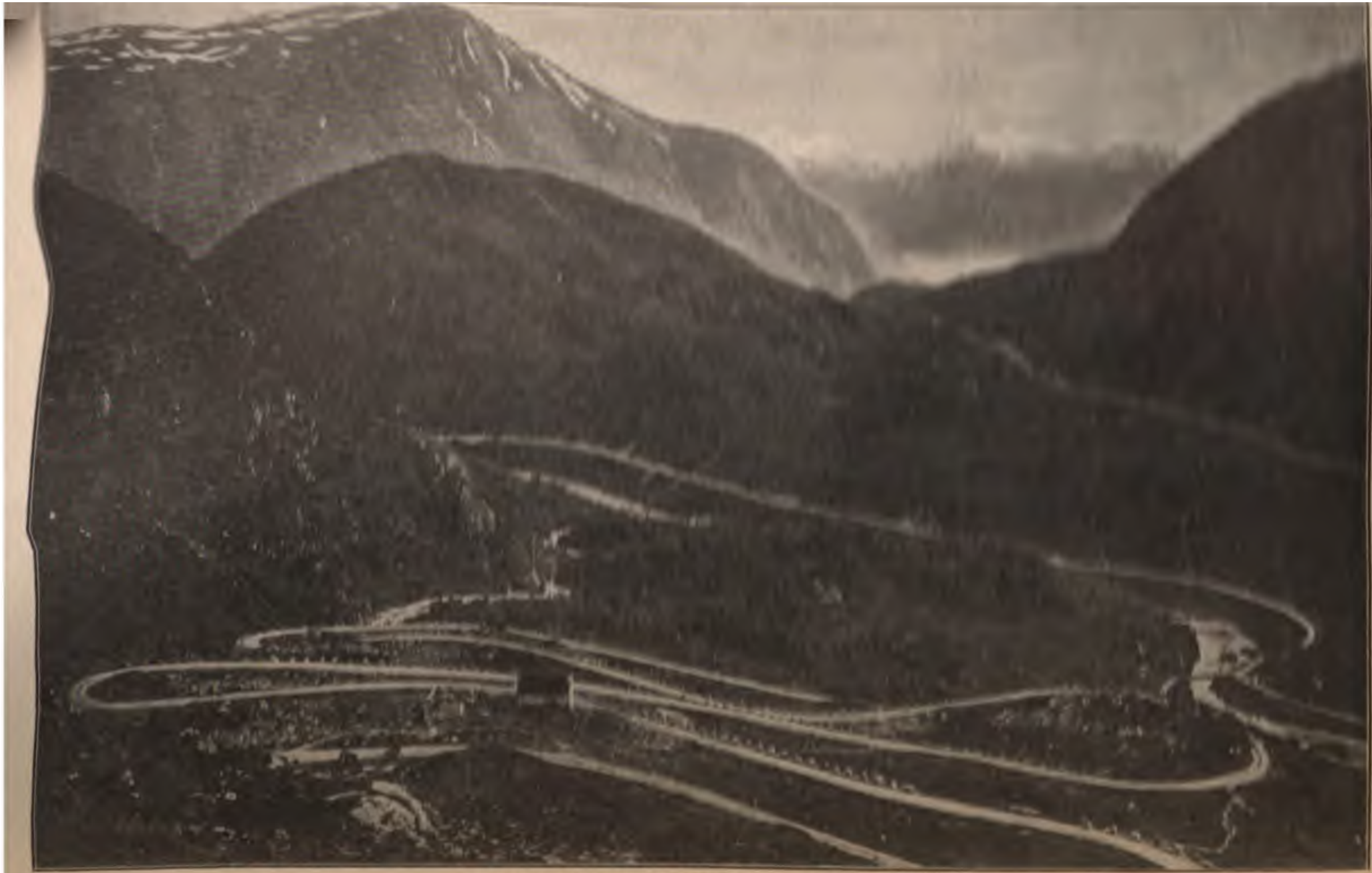

A TYPICAL EXAMPLE OF ROAD BUILDING IN NORWAY

View of the zigzags at Roldal. No other country in the world—except, possibly British India—affords better examples of highway

HIGHWAY CONSTRUCTION

PART II

STREETS AND HIGHWAYS

CITY STREETS

The first work requiring the skill of the engineer is to lay out town sites properly, especially with reference to the future requirements of a large city where any such possibility exists. Few if any of our large cities were so planned. The same principles, to a limited extent, are applicable to all towns or cities. The topography of the site should be carefully studied, and the street lines adapted to it. These lines should be laid out systematically, with a view to convenience and comfort, and also with reference to economy of construction, future sanitary improvements, grades, and drainage.

Arrangement of City Streets. Generally, the best method of laying out streets is in straight lines, with frequent and regular intersecting streets, especially for the business parts of a city. When there is some centrally located structure, such as a courthouse, city hall, market, or other prominent building, it is very desirable to have several diagonal streets leading thereto. In the residence portions of cities, especially if on hilly ground, curves may with advantage replace straight lines, by affording better grades at less cost of grading, and by improving property through avoiding heavy embankments or cuttings.

Width of Streets. The width of streets should be proportioned to the character of the traffic that will use them. No rule can be laid down by which to determine the best width of streets; but it may safely be said that a street which is likely to become a commercial thoroughfare should have a width of not less than 120 feet between the building lines—the carriage-way 80 feet wide, and the sidewalks each 20 feet wide.

In streets occupied entirely by residences a carriage-way 32 feet wide will be ample, but the width between the building lines may be as great as desired. The sidewalks may be any amount over 10 feet

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which fancy dictates. Whatever width is adopted for them, not more of it than 8 feet need be paved, the remainder being occupied with grass and trees.

Street Grades. The grades of city streets depend upon the topography of the site. The necessity of avoiding deep cuttings or high embankments which would seriously affect the value of adjoining property for building purposes, often demands steeper grades than are permissible on country roads. Many cities have paved streets on 20 per cent grades. In establishing grades through unimproved property, they may usually be laid with reference to securing the most desirable percentage within a proper limit of cost. But when improvements have already been made and have been located with reference to the natural surface of the ground, giving a desirable grade is frequently a matter of extreme difficulty without injury to adjoining property. In such cases it becomes a question of how far individual interests shall be sacrificed to the general good. There are, however, certain conditions which it is important to bear in mind:

(1) That the longitudinal crown level should be uniformly sustained from street to street intersection, whenever practicable.

(2) That the grade should be sufficient to drain the surface.

(3) That the crown levels at all intersections should be extended transversely, to avoid forming a depression at the junction.

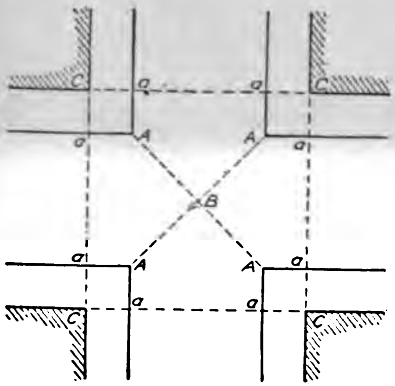


Fig. 47.

Arrangements of Grades at Street Intersections. The best arrangement for intersections of streets when either or both have much inclination, is a matter requiring much consideration, and is one upon which much diversity of opinion exists. No hard or fast rule can be laid down; each will require special adjustment. The best and simplest method is to make the rectangular space *aaaaaaaa*, Fig. 47, level, with a rise of one-half inch in 10 feet from AAAA to B, placing gulleys at AAAA and the catch basins at *ccc*. When this method is not practicable, adopt such a grade (but one not exceeding 2½ per cent) that the rectangle AAAA shall appear to be nearly level; but to secure this it must actually have a considerable dip in the direction of the slope of the street. If steep grades are continued across intersections, they introduce side slopes in the streets thus crossed, which are troublesome, if not dangerous, to vehicles turning the corners, especially the upper ones. Such intersections are especially objectionable in rainy weather. The storm water will fall to the lowest point, concentrating a large quantity of water at two receiving basins, which, with a broken grade, could be divided between four or more basins.

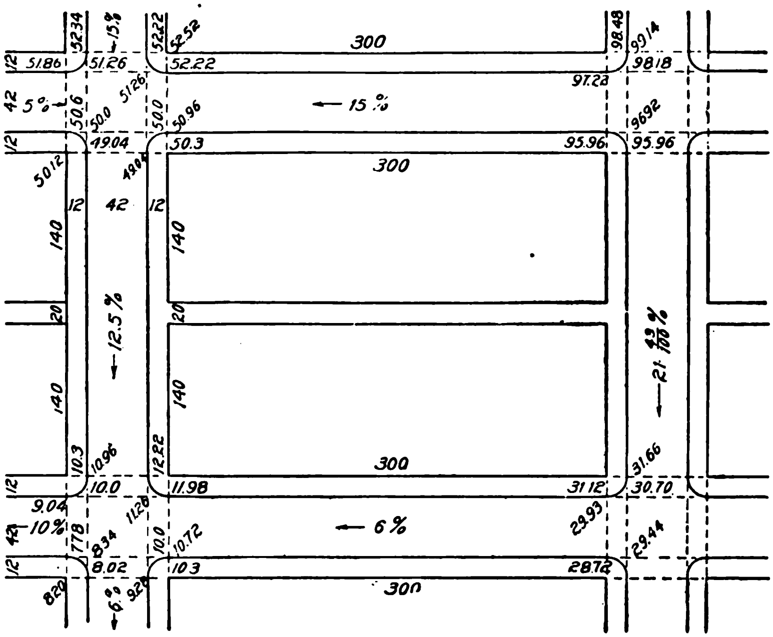


Fig. 48.

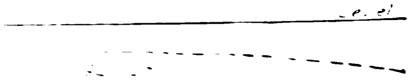
Fig. 48 shows the arrangement of intersections in steep grades adapted for the streets of Duluth, Minn. From this it will be seen that at these intersections the grades are flattened to three per cent for the width of the roadway of the intersecting streets, and that the grade of the curbs is flattened to eight per cent for the width of the intersecting sidewalks. Grades of less amount on roadway or sidewalk are continuous. The elevation of block-corners is found by adding together the curb elevations at the faces of the block-corners, and 2½ per cent of [illegible] at the corner, and dividing [illegible] equal to the average elevation [illegible] plus an average rise of two and one-[illegible] the sidewalk.

[illegible] be introduced between street [illegible] excessive excavation; [illegible] are level or nearly so, for the [illegible] for surface drainage.

[illegible] summits may be calculated [illegible] highest corner; B, the elevation [illegible] distance from corner to corner; [illegible] grade. The elevation of the [illegible]

$$\frac{D \times R + A + B}{2}$$

[illegible] by subtracting the elevation of [illegible] dividing the result by the rate [illegible] mechanically by specially [illegible] to correspond to the rate [illegible] 1 foot per 100 feet, then one [illegible] the map scale. These [illegible] read from [illegible] their figures [illegible] scales read in op-[illegible]



[illegible] at which the opposite [illegible] location of the summits; [illegible] If the difference in [illegible] require an intermediate sum-[illegible] as soon as the scales are placed [illegible]

[illegible] accommodation summit is employed, it should be formed [illegible] joining the two straight grade lines by a vertical curve, as described in Part I. The curve should be used both in the crown of the street and in the curb and footpath.

Where the grade is level between intersections, sufficient fall for surface drainage may be secured without the aid of accommodation summits, by arranging the grades as shown in Fig. 49. The curb is set level between the corners; a summit is formed in the gutter; and receiving basins are placed at each corner.

Transverse Grade. In transverse grade the street should be level; that is, the curbs on opposite sides should be at the same level, and the street crown rise equally from each side to the center. But in hillside streets this condition cannot always be fulfilled, and opposite

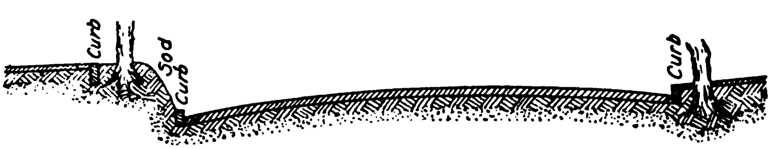


Fig. 50.

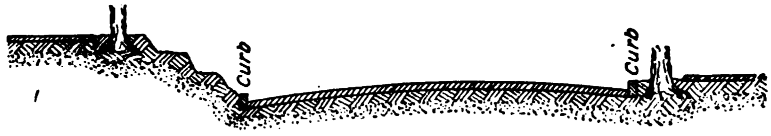


Fig. 51.

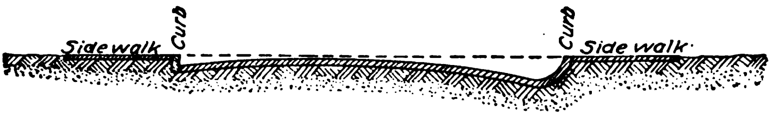


Fig. 52.

sides of the street may differ as much as five feet; in such cases the engineer will have to use his discretion as to whether he shall adopt a straight slope inclining to the lower side, thus draining the whole street by the lower gutter, or adopt the three-curb method and sod the slope of the higher side.

In the improvement of old streets with the sides at different levels, much difficulty will be met, especially where shade trees have to be spared. In such cases, recognized methods have to be abandoned, and the engineer will have to adopt methods of overcoming the difficulties in accordance with the conditions and necessities of each particular case. Figs. 50, 51, and 52 illustrate several typical arrangements in the case of streets in which the opposite sides are at different levels.

Transverse Contour or Crown. The reason for crowning a pavement—*i. e.*, making the center higher than the sides—is to provide for the rapid drainage of the surface. The most suitable form for the crown is the parabolic curve, which may be started at the curb line, or at the edge of the gutter adjoining the carriage-way about one foot from the curb. Fig. 53 shows this form, which is obtained by dividing the ordinate or width from the gutter to the center of the street into ten equal parts, and raising perpendiculars the length of which will be determined by multiplying the rise at the center by the respective number of each perpendicular in the diagram. The amounts thus obtained can be added to the rod readings; and the stakes, set at the proper distance across the street, with their tops at this level, will give the required curve.

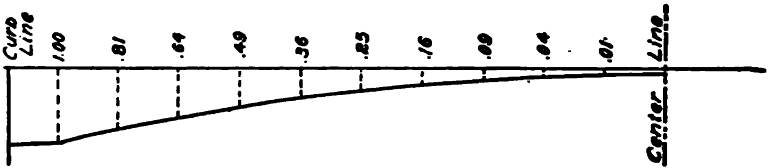


Fig. 53.

The amount of transverse rise, or the height of the center above the gutters, varies with the different paving materials, smooth pavements requiring the least, and rough ones and earth the greatest. The rise is generally stated in a proportion of the width of the carriage-way. The most suitable proportions are:

Stone blocks,	rise	at	center,	$\frac{1}{50}$	width	of	carriage-way.
Wood	"	"	"	$\frac{1}{100}$	"	"	"
Brick	"	"	"	$\frac{1}{50}$	"	"	"
Asphalt	"	"	"	$\frac{1}{50}$	"	"	"

Sub-Foundation Drainage of Streets. The sub-foundation drainage of streets cannot be effected by transverse drains, because of their liability to disturbance by the introduction of gas, water, and other pipes.

Longitudinal drains must be depended upon entirely; they may be constructed of the same materials and in the same manner as road drains. The number of these longitudinal drains must depend upon

STATE HIGHWAY AT WESTWOOD, MASSACHUSETTS

After the tar has been allowed to stand for several hours, sand, fine gravel, or stone screenings are spread thinly over it to absorb the surplus tar which does not penetrate into the road; and the whole roadway is rolled with a heavy roller to smooth the surface.

Courtesy of Massachusetts Highway Commission.

the character of the soil. If the soil is moderately retentive, a single row of tiles or a hollow invert placed under the sewer in the center of the street will generally be sufficient; or two rows of tiles may be employed, one placed outside each curb line; if, on the other hand, the soil is exceedingly wet and the street very wide, four or more lines may be employed. These drains may be permitted to discharge into the sewers of the transverse streets.

Surface Drainage. The removal of water falling on the street surface is provided for by collecting it in the gutters, from which it is discharged into the sewers or other channels by means of catch-basins placed at all street intersections and dips in the street grades.

Gutters. The gutters must be of sufficient depth to retain all the water which reaches them and prevent its overflowing on the footpath. The depth should never be less than 6 inches, and very rarely need be more than 10 inches.

Catch-basins are of various forms, usually circular or rectangular, built of brick masonry coated with a plaster of Portland cement. Whichever form is adopted, they should fulfil the following conditions:

(1) The inlet and outlet should have sufficient capacity to receive and discharge all water reaching the basin.

(2) The basins should have sufficient capacity below the outlet to retain all sand and road detritus, and prevent it being carried into the sewer.

(3) They should be trapped so as to prevent the escape of sewer gas. (This requirement is frequently omitted, to the detriment of the health of the people.)

(4) They should be constructed so that the pit can easily be cleaned out.

(5) The inlet should be so constructed as not easily to be choked by leaves or debris.

(6) They must offer the least possible obstruction to traffic.

(7) The pipe connecting the basin to the sewer should be easily freed of any obstruction.

The bottom of the basins should be 6 or 8 feet below the street level; and the water level in them should be from 3 to 4 feet lower than the street surface, as a protection against freezing.

The capacity and number of basins will depend upon the area of surface which they drain

In streets having level or light longitudinal grades, gullies may be constructed along the line of the gutter at such intervals as may be found necessary.

Catch-basins are usually placed at the curb line. In several cities, the basin is placed in the center of the street, and connects to inlets placed at the curb line. This reduces the cost of construction and cleaning, and removes from the sidewalk the dirty operations of cleaning the basins.

Catch-basins and gully-pits require to be cleaned out at frequent intervals; otherwise the odor arising from the decomposing matter contained in them will be very offensive. No rule can be laid down for the intervals at which the cleaning should be done, but they must be cleaned often enough to prevent the matter in them from putrefying. There is no uniformity of practice observed by cities in this matter; in some, the cleaning is done but once a year; in others, after every rainstorm; in still others, at intervals of three or four months; while in a few cities the basins are cleaned out once a month.

FOUNDATIONS

The stability, permanence, and maintenance of any pavement depend upon its foundation. If the foundation is weak, the surface will soon settle unequally, forming depressions and ruts. With a good [illegible]

[illegible] will require its own special treatment. Whatever its character, it must be brought to a dry and tolerably hard condition by draining and rolling. Sand and gravels which do not hold water, present no difficulty in securing a solid and secure foundation; clays and soils retentive of water are the most difficult. Clay should be excavated to a depth of at least 18 inches below the surface of the finished covering; and the space so excavated should be filled in with sand, furnace slag, ashes, coal dust, oyster shells, broken brick, or other materials which are not excessively absorbent of water. A clay soil or one retaining water may be cheaply and effectually improved by laying cross-drains with open joints at intervals of 50 or 100 feet. These drains should be not less than 18 inches below the surface, and the trenches filled with gravel. They should be 4 inches in internal diameter, and should empty into longitudinal drains.

Sand and planks, gravel, and broken stone have been successively used to form the foundation for pavements; but, although eminently useful materials, their application to this purpose has always been a failure. Being inherently weak and possessing no cohesion, the main reliance for both strength and wear must be placed upon the surface-covering. This covering—usually (except in case of sheet asphalt) composed of small units, with joints between them varying from one-half an inch to one and a-half inches—possesses no elements of cohesion; and under the blows and vibrations of traffic the independent units or blocks will settle and be jarred loose. On account of their porous nature, the subsoil quickly becomes saturated with urine and surface waters, which percolate through the joints; winter frosts upheave them; and the surface of the street becomes blistered and broken up in dozens of places.

Concrete. As a foundation for all classes of pavement (broken stone excepted), hydraulic-cement concrete is superior to any other. When properly constituted and laid, it becomes a solid, coherent mass capable of bearing great weight without crushing. If it fail at all, it must fail altogether. The concrete foundation is the most costly, but this is balanced by its permanence and by the saving in the cost of repairs to the pavement which it supports. It admits of access to subterranean pipes with less injury to the neighboring pavement than any other, for the concrete may be broken through at any point without unsettling the foundation for a considerable distance around it, as is the case with sand or other incoherent material; and when the concrete is replaced and set, the covering may be reset at its proper level, without the uncertain allowance for settlement which is necessary in other cases.

Thickness of Concrete. The thickness of the concrete bed must be proportioned by the engineer; it should be sufficient to provide against breaking under transverse strain caused by the settlement of the subsoil. On a well-drained soil, six inches will be found sufficient; but in moist and clayey soils, twelve inches will not be excessive. On such soils a layer of sand or gravel, spread and compacted before placing the concrete, will be found very beneficial.

The proportions of the ingredients for concrete used for pavement foundations are usually:

1 part Portland cement
3 parts Sand
7 parts Broken Stone.

Or,

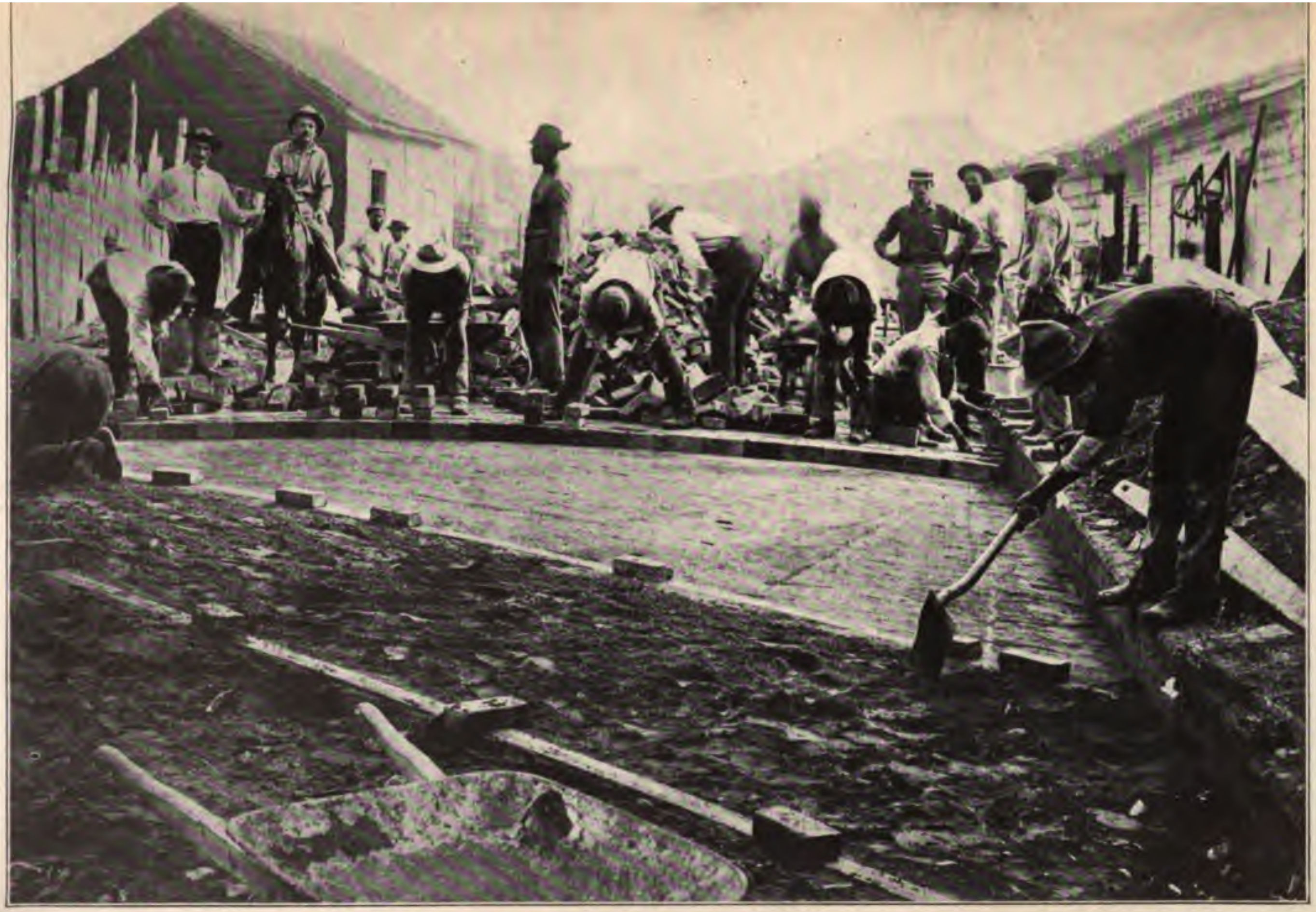
1 part Natural Hydraulic Cement
2 parts Sand
5 parts Broken Stone.

The question is sometimes raised as to whether Natural or Portland cement should be used. Natural cement is more extensively employed on account of its being cheaper in price than Portland. There is no advantage gained in using Portland cement. Concrete should not be laid when the temperature falls below 32° F.

The concrete foundation, after completion, should be allowed to remain several days before the pavement is placed upon it, in order that the mortar may become entirely set. During setting, the concrete should be protected from the drying action of the sun and wind, and should be kept damp to prevent the formation of drying cracks.

STONE BLOCK PAVEMENTS

Stone blocks are commonly employed for pavements where traffic is heavy. The material of which the blocks are made should possess sufficient hardness to resist the abrasive action of traffic, and sufficient toughness to prevent them from being broken by the impact of loaded wheels. The hardest stones will not necessarily give the best results in the pavement, since a very hard stone usually wears smooth and becomes slippery. The edges of the block chip off, and the



REPAVING A STREET IN THE CITY OF PANAMA WITH VITRIFIED BRICK LAID UPON CONCRETE

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ASTOR, LENOX
TILDEN FOUNDATIONS

upper face becomes rounded, thus making the pavement very rough.

The stone is sometimes tested to determine its strength, resistance to abrasion, etc.; but, as the conditions of use are quite different from those under which it may be tested, such tests are seldom satisfactory. However, examination of a stone as to its structure, the closeness of its grain, its homogeneity, porosity, etc., may assist in forming an idea of its value for use in a pavement. A low degree of permeability usually indicates that the material will not be greatly affected by frost.

Materials.—Granite. Granite is more extensively employed for stone block paving than any other variety of stone; and because of this fact, the term "granite paving" is generally used as being synonymous with stone block paving. The granite employed should be of a tough, homogeneous nature. The hard, quartz granites are usually brittle, and do not wear well under the blows of horses' feet or the impact of vehicles; granite containing a high percentage of feldspar will be injuriously affected by atmospheric changes; and granite in which mica predominates will wear rapidly on account of its laminated structure. Granite possesses the very important property of splitting in three planes at right angles to one another, so that paving blocks may readily be formed with nearly plane faces and square corners. This property is called the *rift* or *cleavage*.

Sandstones of a close-grained, compact nature often give very satisfactory results under heavy traffic. They are less hard than granite, and wear more rapidly, but do not become smooth and slippery. Sandstones are generally known in the market by the name of the quarry or place where produced as "Medina," "Berea," etc.

Trap rock, while answering well the requirements as to durability and resistance to wear, is objectionable on account of its tendency to wear smooth and become slippery; it is also difficult to break into regular shapes.

Limestone has not usually been successful in use for the construction of block pavements, on account of its lack of durability against atmospheric influences. The action of frost commonly splits the blocks; and traffic shivers them, owing to the lamination being vertical.

TABLE [illegible]
Specific Gravity, Weight, Resistance to Crushing, and Absorption Power of Stones.

MATERIAL	SPECIFIC GRAVITY	WEIGHT Pounds per cu. ft.	RESISTANCE TO CRUSHING Pounds per sq. in.	PERCENTAGE OF WATER ABSORBED
[illegible]				
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]				
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]				
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]				
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]				
[illegible]	[illegible]		[illegible]	
[illegible]	[illegible]		[illegible]	

Cobblestone Pavement. Cobblestones bedded in sand possess the merit of cheapness, and afford an excellent foothold for horses; but the roughness of such pavements requires the expenditure of a large amount of tractive energy to move a load over them. Aside from this, cobblestones are entirely wanting in the essential requisites of a good pavement. The stones being of irregular size, it is almost imposdisplaced by the introduction of rectangular blocks of granite. Blocks of comparatively large dimensions were at first employed. They were from 6 to 8 inches in width on the surface, from 10 to 20 inches in length, with a depth of 9 inches. They were merely placed in rows on the subsoil, perfunctorily rammed, the joints filled with sand, and the street thrown open to traffic. The unequal settlement of the blocks, the insufficiency of the foothold, and the difficulty of cleansing the street, led to the gradual development of the latest type of stone-block pavement, which consists of narrow, rectangular blocks of granite, properly proportioned, laid on an unyielding and impervious foundation, with the joints between the blocks filled with an impermeable cement.

Experience has proved beyond doubt that this latter type of pavement is the most enduring and economical for roadways subjected to heavy and constant traffic. Its advantages are many, while its defects are few.

Advantages.

(1) Adaptability to all grades.
(2) Suits all classes of traffic.
(3) Exceedingly durable.
(4) Foothold, fair.
(5) Requires but little repair.
(6) Yields but little dust or mud.
(7) Facility for cleansing, fair.

Defects.

(1) Under certain conditions of the atmosphere, the surface of the pavement becomes greasy and slippery.

(2) The incessant din and clatter occasioned by the movement of traffic is an intolerable nuisance; it is claimed by many physicians that the noise injuriously affects the nerves and health of persons who are obliged to live or do business in the vicinity of streets so paved.

(3) Horses constantly employed upon it soon suffer from the continual jarring produced in their legs and hoofs, and quickly wear out.

(4) The discomfort of persons riding over the pavement is very great, because of the continual jolting to which they are subjected.

(5) If stones of an unsuitable quality are used—for example,

those that polish—the surface quickly becomes slippery and exceedingly unsafe for travel.

Size and Shape of Blocks. The proper size of blocks for paving purposes has been a subject of much discussion, and a great variety of forms and dimensions are to be found in all cities.

For stability, a certain proportion must exist between the depth, the length, and the breadth. The depth must be such that when the wheel of a loaded vehicle passes over one edge of the upper surface of a block, the block will not tend to tip up. The resultant direction of the pressure of the load and adjoining blocks should always tend to depress the whole block vertically; where this does not happen, the maintenance of a uniform surface is impossible. To fulfil this requirement, it is not necessary to make the block more than six inches deep.

Width of Blocks. The maximum width of blocks is controlled by the size of horses' hoofs. To afford good foothold to horses drawing heavy loads, it is necessary that the width of each block, measured along the street, shall be the least possible consistent with stability.

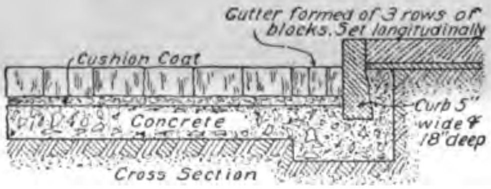

Fig. 54.

If the width be great, a horse drawing a heavy load, attempting to find a joint, slips back, and requires an exceptionally wide joint to pull him up. It is therefore desirable that the width of a block shall not exceed 3 inches; or that four blocks, taken at random and placed side by side, shall not measure more than 14 inches.

Length of Blocks. The length, measured across the street, must be sufficient to break joints properly, for two or more joints in line lead to the formation of grooves. For this purpose the length of the block should be not less than 9 inches nor more than 12 inches.

Form of Blocks. The blocks should be well squared, and must not taper in any direction; sides and ends should be free from irregular projections. Blocks that taper from the surface downwards (wedge-shaped) should not be permitted in the work; but if any are allowed, they should be set with the widest side down.

Manner of Laying Blocks. The blocks should be laid in parallel courses, with their longest side at right angles to the axis of the street, and the longitudinal joints broken by a lap of at least two inches (see Figs. 54 and 55). The reason for this is to prevent the formation of longitudinal ruts, which would happen if the blocks were laid lengthwise. Laying blocks obliquely and "herring-bone" fashion has been tried in several cities, with the idea that the wear and formation of ruts would be reduced by having the vehicle cross the blocks diagonally. The method has failed to give satisfactory results; the wear was irregular and the foothold defective; the difficulty of construction was increased by reason of labor required to form the triangular joints; and the method was wasteful of material.

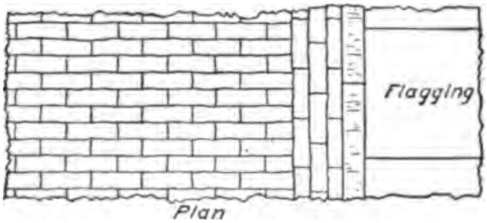

Fig. 55.

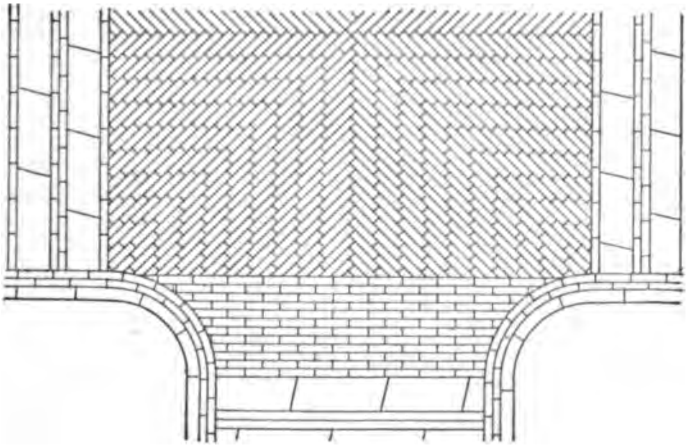

Fig. 56.

The gutters should be formed by three or more courses of block, laid with their length parallel to the curb.

At junctions or intersections of streets, the blocks should be laid diagonally from the center, as shown in Fig. 56. The reasons for this are: (1) To prevent the traffic crossing the intersection from following the longitudinal joints and thus forming depressions and ruts; (2) Laid in this manner, the blocks afford a more secure foothold for horses turning the corners. The ends of the diagonal blocks where they abut against the straight blocks, must be cut to the required bevel.

The blocks forming each course must be of the same depth, and no deviation greater than one-quarter of an inch should be permitted. The blocks should be assorted as they are delivered, and only those corresponding in depth and width should be used in the same course. The better method would be to gauge the blocks at the quarry. This would lessen the cost considerably; it would also avoid the inconvenience to the public due to the stopping of travel because of the rejection of defective material on the ground. This method would undoubtedly be preferable to the contractor, who would be saved the expense of handling unsatisfactory material; and it would also leave the inspectors free to pay more attention to the manner in which the work of paving is performed.

The accurate gauging of the blocks is a matter of much importance. If good work is to be executed, the blocks, when laid, must be in parallel and even courses; and if the blocks be not accurately gauged to one uniform size, the result will be a badly paved street, with the courses running unevenly. The cost of assorting blocks into lots of uniform width, after delivery on the street, is far in excess of any additional price which would have to be paid for accurate gauging at the quarry.

Foundation. The foundation of the blocks must be solid and unyielding. A bed of hydraulic-cement concrete is the most suitable, the thickness of which must be regulated according to the traffic; the thickness, however, should not be less than 4 inches, and need not be more than 9 inches. A thickness of 6 inches will sustain traffic of 600 tons per foot of width.

Cushion Coat. Between the surface of the concrete and the base of the blocks, there must be placed a cushion-coat formed of an incompressible but mobile material, the particles of which will readily adjust themselves to the irregularities of the bases of the blocks and transfer the pressure of the traffic uniformly to the concrete below. A layer of dry, clean sand 1 to 2 inches thick forms an excellent cushion-coat.

Its particles must be of such fineness as to pass through a No. 8 screen; if coarse and containing pebbles, they will not adapt themselves to the irregularities of the bases of the blocks; hence the blocks will be supported at only a few points, and unequal settlement will take place when the pavement is subjected to the action of traffic. The sand must also be perfectly free from moisture, and artificial heat must be used to dry it if necessary. This requirement is an absolute necessity. There should be no moisture below the blocks when laid; nor should water be allowed to penetrate below the blocks; if such happens, the effect of frost will be to upheave the pavement and crack the concrete.

Where the best is desired without regard to cost, a layer half an inch thick of asphaltic cement may be substituted for the sand, with superior and very satisfactory results.

Laying Blocks. The blocks should be laid stone to stone, so that the joint may be of the least possible width; wide joints cause increased wear and noise, and do not increase the foothold. The courses should be commenced on each side and worked toward the middle; and the last stone should fit tightly.

Ramming. After the blocks have been set, they should be well rammed down; and the stones which sink below the general level should be taken up and replaced with a deeper stone or brought to level by increasing the sand bedding.

The practice of workmen is invariably to use the rammer so as to secure a fair surface. This is not the result intended to be secured, but to bring each block to an unyielding bearing. The result of such a surfacing process is to produce an unsightly and uneven roadway when the pressure of traffic is brought upon it. The rammer used should weigh not less than 50 pounds and have a diameter of not less than 3 inches.

Joint Filling. All stone block pavements depend for their waterproof qualities upon the character of the joint filling. Joints filled with sand and gravel are of course pervious. A grout of lime or cement mortar does not make a permanently waterproof joint; it becomes disintegrated under the vibration of traffic. An impervious joint can be made only by employing a filling made from bituminous or asphaltic material; this renders the pavement more impervious to moisture, makes it less noisy, and adds considerably to its strength.

Bituminous Cement for Joint Filling. The bituminous materials employed are: (1) The tar produced in the manufacture of gas, which, when redistilled, is called *distillate*, and is numbered 1, 2, 3, 4, etc., according to its density; this material under the name of paving-pitch is extensively used, both alone and in combination with other bituminous substances; (2) Combinations of gas tar or coal tar with refined asphaltum; (3) Mixtures of refined asphaltum, creosote, and coal tar.

The formula for the bituminous joint filling used in New York City, is:

Refined Trinidad asphaltum	20 parts.
No. 4 coal-tar distillate	100 parts.
Residuum of petroleum	3 parts.

In Washington, D. C., coal tar distillate No. 6 is used alone.

In Europe a bituminous cement much used is composed of coal-tar, asphaltum, gas tar, and creosote oil, in the proportion of 100 pounds of asphaltum to 4 gallons of tar and 1 gallon of creosote. These proportions are varied somewhat, according to the quality of the asphaltum employed. The mixture is melted, and is boiled from one to two hours in a suitable boiler, being then poured into the joints in a boiling state. This mixture is impervious to moisture, and possesses a degree of elasticity sufficient to prevent it from cracking.

The mode of applying the bituminous cement is as follows: After the blocks are rammed, the joints are filled to a depth of about two inches with clean gravel heated to a temperature of about 250° F.; then the hot cement is poured in until it forms a layer of about one inch on top of the gravel; then more gravel is filled in to a depth of about two inches; then cement is poured in until it appears on top of the gravel, more gravel being next added until it reaches to within half an inch of the top of the blocks; this remaining half-inch is filled with cement, and then fine gravel or sand is sprinkled over the joints.

In some cases the joints are first filled with heated gravel; the cement is poured in until the sand beneath and the gravel between the blocks will absorb no more, and the joints are filled flush with the top of the pavement. This method is open to objection; for, if the gravel is not sufficiently hot, the cement will be chilled and will not flow to the bottom of the joint, but, instead, will form a thin layer near the surface, which under the action of frost and the vibration of traffic, will be quickly cracked and broken up; the gravel will settle, and the

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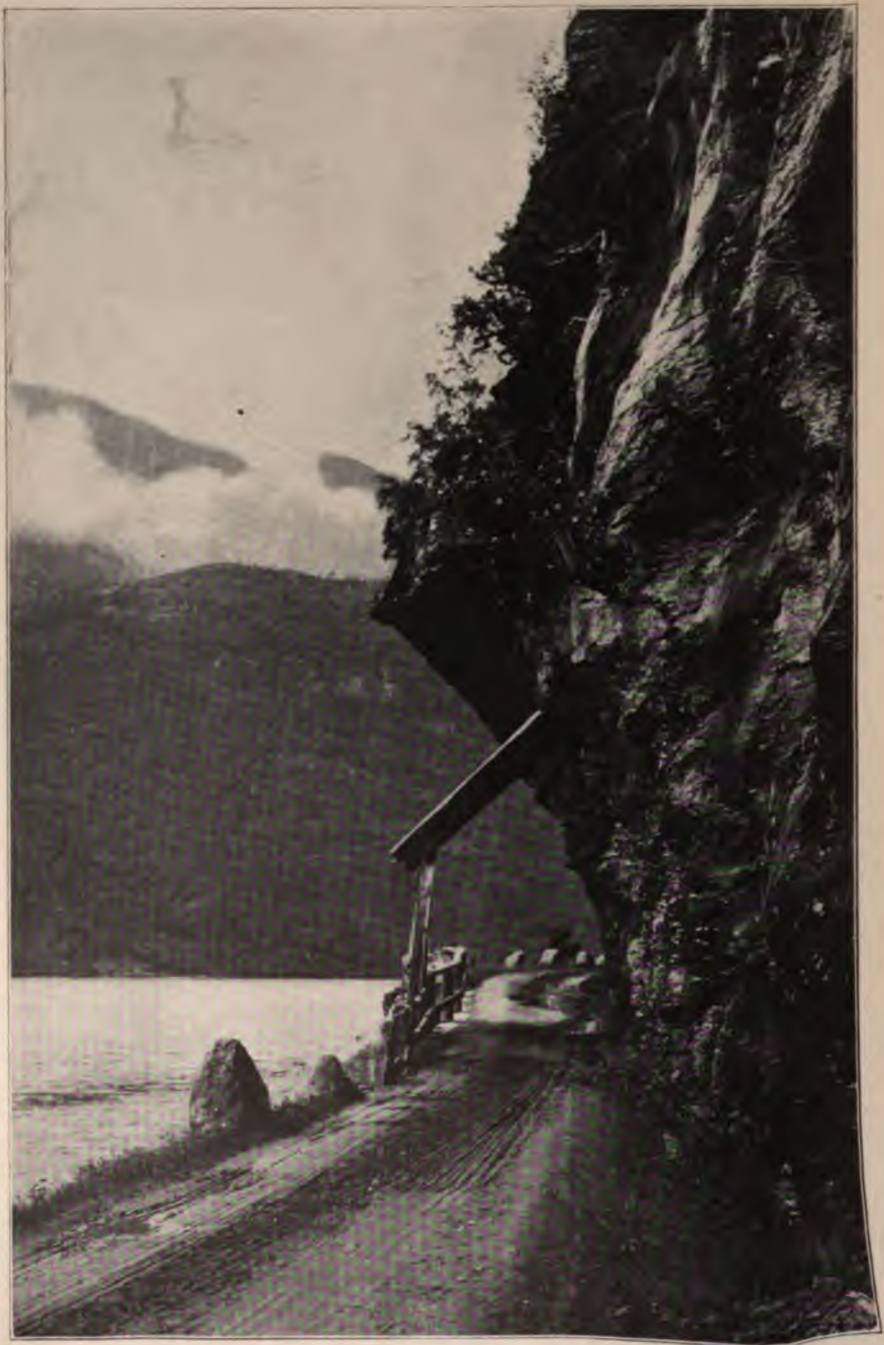


A GOOD EXAMPLE OF ROAD-BUILDING ON A STEEP, ROCKY HILLSIDE
View near Eide, on Hardanger Fjord, Norway.

blocks will be jarred loose, the surface of the pavement becoming a series of ridges and hollows.

The quantity of cement required per square yard of pavement will vary according to the shape of the blocks, the width of the joints, and the depth of the sand bed. With well-shaped blocks, close joints, and a half-inch sand bed, the quantity will vary from 3½ to 5 gallons; with ill-shaped blocks, wide joints, and a heavy sand bed, 10 to 12 gallons would not be an excessive amount to use to secure the result obtained by employing well-shaped blocks and close joints.

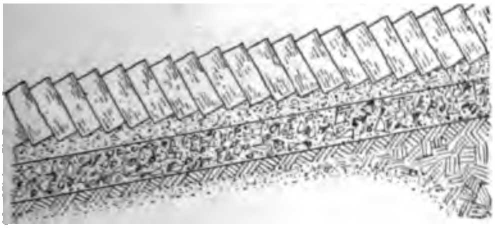


Fig. 57.

Stone Pavement on Steep Grades. Stone blocks may be employed on all practicable grades; but on grades exceeding 10 per cent, cobblestones afford a better foothold than blocks. The cobblestones should be of uniform length, the length being at least twice the breadth —say stones 6 inches long and 2½ to 3 inches in diameter. These should be set on a concrete foundation, laid stone to stone, and the interstices filled with cement grout or bituminous cement; or a bituminous concrete foundation may be employed and the interstices between the stones filled with asphaltic paving cement. Should stone blocks be preferred, they must be laid, when the grade exceeds 5 per cent, with a serrated surface, by either of the methods shown in Figs. 57 and 58. The method shown in Fig. 57 consists in slightly tilting the blocks on their bed so as to form a series of ledges or steps, against which the horses' feet being planted, a secure foothold is obtained. The method shown in Fig. 58 consists in placing between the rows of

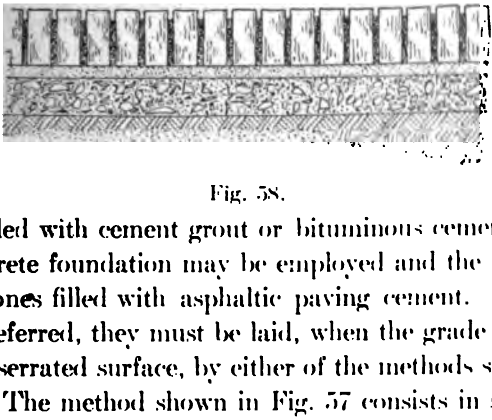


Fig. 58.

stones a course of slate, or strips of creosoted wood, rather less than one inch in thickness and about an inch less in depth than the blocks; or the blocks may be spaced about one inch apart, and the joints filled with a grout composed of gravel and cement. The pebbles of the gravel should vary in size between one-quarter and three-quarters of an inch.

BRICK PAVEMENTS

Characteristics of Brick Suitable for Paving. These are:

(1) Not to be acted upon by acids.

(2) Not to absorb more than 1-600 of its weight of water in 48 hours.

(3) Not susceptible to polish.

(4) Rough to the touch, resembling fine sandpaper.

(5) To give a clear, ringing sound when struck together.

(6) When broken, to show a compact, uniform, close-grained, structure, free from air-holes and pebbles.

(7) Not to scale, spall, or chip when quickly struck on the edges.

(8) Hard but not brittle.

Tests of Paving Brick. To ascertain the quality of paving brick, they are generally subjected to four tests, namely: (1) Abrasion by impact (commonly called the "Rattler" test); (2) absorption; (3) transverse or cross-breaking; (4) crushing. With a view to securing uniformity in the methods of making the above tests, the National Brick Manufacturers' Association has adopted and recommends the following:

Rattler Test

1. *Dimensions of the Machine.* The standard machine shall be 28 inches in diameter and 20 inches in length, measured inside the chamber.

Other machines may be used, varying in diameter between 26 and 30 inches, and in length between 18 and 24 inches; but if this is done, a record of it must be attached to the official report. Long rattlers may be cut up into sections of suitable length by the insertion of an iron diaphragm at the proper point.

2. *Construction of the Machine.* The barrel shall be supported on trunnions at either end; in no case shall a shaft pass through the rattling chamber. The cross-section of the barrel shall be a regular polygon having 14 sides. The heads shall be composed of gray cast-

iron, not chilled or case-hardened. The staves shall preferably be composed of steel plates, since cast-iron peans and ultimately breaks under the wearing action on the inside. There shall be a space of one-fourth of an inch between the staves, for the escape of dust and small pieces of waste. Other machines may be used, having from twelve to sixteen staves; but if this is done, a record of it must be attached to the official report of the test.

3. *Composition of the Charge.* All tests must be executed on charges containing but one make of brick or block at a time. The charge shall consist of 9 paving blocks or 12 paving bricks, together with 300 pounds of shot made of ordinary machinery cast-iron. This shot shall be of two sizes, as described below; and the shot charge shall be composed one-fourth (75 pounds) of the larger size, and three-fourths (225 pounds) of the smaller size.

4. *Size of the Shot.* The larger size shall weigh about 7½ pounds and be about 2½ inches square and 4½ inches long, with slightly rounded edges. The smaller size shall be cubes of 1½ inches on a side, with rounded edges. The individual shot shall be replaced by new ones when they have lost one-tenth of their original weight.

5. *Revolutions of the Charge.* The number of revolutions of a standard test shall be 1,800; and the speed of rotation shall not fall below 28 nor exceed 30 revolutions per minute. The belt-power shall be sufficient to rotate the rattler at the same speed, whether charged or empty.

6. *Condition of the Charge.* The bricks composing a charge shall be thoroughly dried before making the test.

7. *Calculation of the Results.* The loss shall be calculated in per cents of the weight of the dry brick composing the charge; and no result shall be considered as official unless it is the average of two distinct and complete tests made on separate charges of brick.

Absorption Test

1. The number of bricks for a standard test shall be five.
2. The test must be conducted on rattled brick. If none such are available, the whole brick must be broken in halves before treatment.
3. Dry the bricks for 48 hours at a temperature ranging from 230° to 250° F. before weighing for the official dry weight.
4. Soak for 48 hours completely immersed in pure water.

5. After soaking, and before weighing, the bricks must be wiped dry from surplus water.

6. The difference in the weight must be determined on scales sensitive to one gram.

7. The increase in weight due to water absorbed shall be calculated in per cents of the initial dry weight.

Cross-Breaking Test

1. Support the brick on edge, or as laid in the pavement, on hardened steel knife-edges, rounded longitudinally to a radius of twelve inches and transversely to a radius of one-eighth inch, and bolted in position so as to secure a span of six inches.

2. Apply the load to the middle of the top face through a hardened steel knife-edge, straight longitudinally and rounded transversely to a radius of one-sixteenth inch.

3. Apply the load at a uniform rate of increase till fracture ensues.

4. Compute the modulus of rupture by the formula

$$f=\frac{3wl}{2bd^2},$$

in which f = modulus of rupture, in pounds per square inch;
w = total breaking load, in pounds;
l = length of span, in inches = 6;
b = breadth of brick, in inches;
d = depth of brick, in inches.

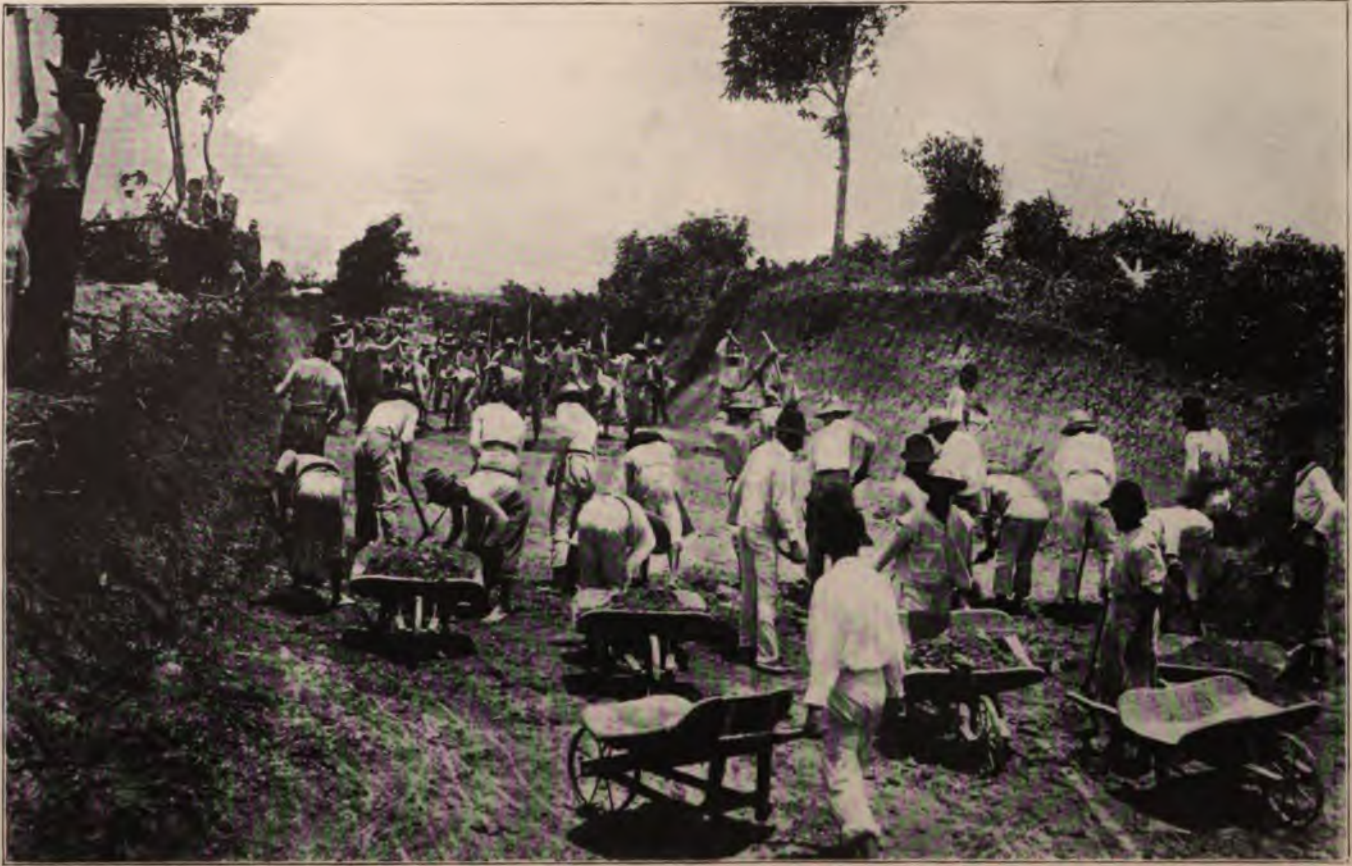
5. Samples for test must be free from all visible irregularities of surface or deformities of shape, and their upper and lower faces must be practically parallel.

6. Not less than ten brick shall be broken, and the average of all shall be taken for a standard test.

Crushing Test

1. The crushing test should be made on half-bricks, loaded edgewise, or as they are laid in the street. If the machine used is unable to crush a full half-brick, the area may be reduced by chipping off, keeping the form of the piece to be tested as nearly prismatic as possible. A machine of at least 100,000 pounds' capacity should be used; and the specimen should not be reduced below four square inches of area in cross-section at right angles to direction of load.

2. The upper and lower surfaces should preferably be ground to



ROAD-BUILDING IN PORTO RICO

Earth cut in course of excavation on road between Caguas and Humacao.

true and parallel planes. If this is not done, they should be bedded, while in the testing machine, in plaster of Paris, which should be allowed to harden ten minutes under weight of the crushing planes only, before the load is applied.

3. The load should be applied at a uniform rate of increase to the point of rupture.

4. Not less than an average obtained from five tests on five different bricks shall constitute a standard test.

Properties of Paving Bricks. Paving bricks range in weight from 5½ to 7½ pounds; in specific gravity, from 1.91 to 2.70; in resistance to crushing, from 7,000 to 18,000 pounds per square inch; in resistance to cross-breaking, R = 1,400 to 2,000 pounds; in absorption, from 0.15 to 3 per cent in 24 hours. The dimensions vary according to locality and the requirements of the specifications. The "standard" bricks are 2½ × 4 × 8 inches, requiring 58 bricks to the square yard, and weighing 7 pounds each; "repressed", 2½ × 4 × 8½ inches, requiring 61 to the square yard, and weighing 6½ pounds each; "Metropolitan", 3 × 4 × 9 inches, requiring 45 to the square yard, and weighing 9½ pounds each.

Advantages of Brick Pavements. These may be stated as follows:

(1) Ease of traction.
(2) Good foothold for horses.
(3) Not disagreeably noisy.
(4) Yields but little dust and mud.
(5) Adapted to all grades.
(6) Easily repaired.
(7) Easily cleaned.
(8) But slightly absorbent.
(9) Pleasing to the eye.
(10) Expeditiously laid.
(11) Durable under moderate traffic.

Defects of Brick Pavements. The principal defects of brick pavements arise from lack of uniformity in the quality of the bricks, and from the liability of incorporating in the pavement bricks of too soft or porous a structure, which crumbles under the action of traffic or frost.

Foundation. A brick pavement should have a firm foundation. As the surface is made up of small, independent blocks, each one must be adequately supported, or the load coming upon it may force it downwards and cause unevenness, a condition which conduces to the rapid destruction of the pavement. Several forms of foundation have been used—such as gravel, plank, sand, broken stone, and concrete. The last mentioned is doubtless the best.

Sand Cushion. The sand cushion is a layer of sand placed on top of the concrete to form a bed for the brick. Practice regarding the depth of this layer of sand varies considerably. In some cases it is only half an inch deep, varying from this up to three inches. The sand cushion is very desirable, as it not only forms a perfectly true and even surface upon which to place brick, but also makes the pavement less hard and rigid than would be the case were the brick laid directly on the concrete.

The sand is spread evenly, sprinkled with water, smoothed, and brought to the proper contour by screeds or wooden templets, properly trussed, mounted on wheels or shoes which bear upon the upper surface of the curb. Moving the templet forward levels and forms the sand to a uniform surface and proper shape.

The sand used for the cushion-coat should be clean and free from loam, moderately coarse, and free from pebbles exceeding one-quarter inch in size.

Manner of Laying. The bricks should be laid on edge, as closely and compactly as possible, in straight courses across the street, with the length of the bricks at right angles to the axis of the street. Joints should be broken by at least 3 inches. None but whole bricks should be used, except in starting a course or making a closure. To provide for the expansion of the pavement, both longitudinal and transverse expansion-joints are used, the former being made by placing a board templet seven-eighths of an inch thick against the curb and abutting the brick thereto. The transverse joints are formed at intervals varying between 25 and 50 feet, by placing a templet or building-lath three-eighths of an inch thick between two or three rows of brick. After the bricks are rammed and ready for grouting, these templets are removed, and the spaces so left are filled with coal-tar pitch or asphaltic paving cement. The amount of pitch or cement required will vary between one and one and a-half pounds per square yard of pavement, depending upon the width of the joints. After 25 or 30 feet of the pavement is laid, every part of it should be rammed with a rammer weighing not less than 50 pounds; and the bricks which sink below the general level should be removed, sufficient sand being added to raise the brick to the required level. After all objectionable brick have been removed, the surface should be swept clean, then rolled with a steam roller weighing from 3 to 6 tons. The object of rolling is to bring the bricks to an unyielding bearing with a plane surface; if this is not done, the pavement will be rough and noisy and will lack durability. The rolling should be first executed longitudinally, beginning at the crown and working toward the gutter, taking care that each return trip of the roller covers exactly the same area as the preceding trip, so that the second passage may neutralize any careening of the brick due to the first passage.

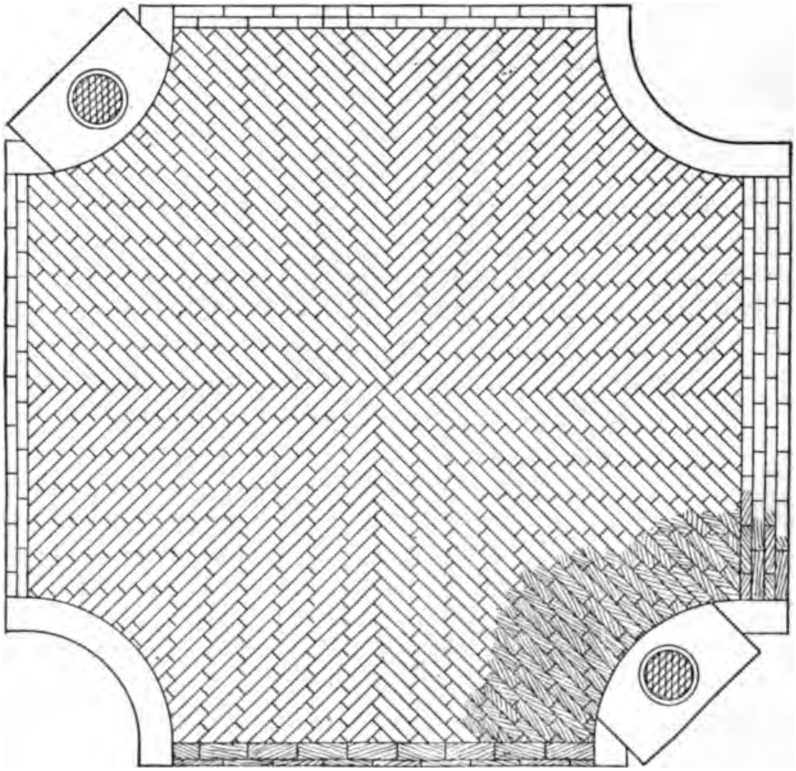


Fig. 59.

The manner of laying brick at street intersections is shown in Fig. 59.

Joint Filling. The character of the material used in filling the joints between the brick has considerable influence on the success and durability of the pavement. Various materials have been used—such as sand, coal-tar pitch, asphalt, mixtures of coal-tar and asphalt, and Portland cement, besides various patented fillers, as "Murphy's grout", which is made from ground slag and cement. Each material has its advocates, and there is much difference of opinion as to which gives the best results.

The best results seem to be obtained by using a high grade of Portland cement containing the smallest amount of lime in its composition, the presence of the lime increasing the tendency of the filler to swell through absorption of moisture, causing the pavement to rise or to be lifted away from its foundation, and thus producing the roaring or rumbling noise so frequently complained of.

The Portland cement grout, when uniformly mixed and carefully placed, resists the impact of traffic and wears well with brick. When a failure occurs, repairs can be made quickly; and, if made early, the pavement will be restored to a good condition. If, however, repairs are neglected, the brick soon loosens and the pavement fails.

The office of a filler is to prevent water from reaching the foundation, and to protect the edges of the brick from spalling under traffic. In order to meet both of these requirements, every joint must be filled to the top, and must remain so, wearing down with the brick. Sand does not meet these requirements. Although at first making a good filler, being inexpensive and reducing the liability of the pavement to be noisy, it soon washes out, leaving the edges of the brick unprotected and consequently liable to be chipped. Coal-tar and the mixtures of coal-tar and asphalt have an advantage in rendering a pavement less noisy and in cementing together any breaks that may occur through upheavals from frost or other causes; but, unless made very hard, they have the disadvantage of becoming soft in hot weather and flowing to the gutters and low places in the pavement, there forming a black and unsightly scale and leaving the high parts unprotected. The joints, thus deprived of their filling, become receptacles for water, mud, and ice in turn; and the edges of the brick are quickly broken down. Some of these mixtures become so brittle in winter that they crack and fly out of the joints under the action of traffic.

The Portland cement filler is prepared by mixing two parts of cement and one part of fine sand with sufficient water to make a thin grout. The most convenient arrangement for preparing and distributing the grout is a water-tight wooden box carried on four wooden wheels about 12 inches in diameter. The box may be about 4 feet wide, 7 feet long, and 12 inches deep, furnished with a gate about 8 inches wide, in the rear end. The box should be mounted on the wheels with an inclination, so that the rear end is about 4 inches lower than the front end.

The operation of placing the filler is as follows: The cement and sand are placed in the box, and sufficient water is added to make a thin grout. The box is located about 12 feet from the gutter, the end gate opened, and about 2 cubic feet of the grout allowed to flow out and run over the top of the brick (care being taken to stir the grout while it is being discharged). If the brick are very dry, the entire surface of the pavement should be thoroughly wet with a hose before applying the grout; if not, absorption of the water from the grout by the bricks will prevent adhesion between the bricks and the cement grout. The grout is swept into the joints by ordinary bass brooms. After about 100 feet in length of the pavement has been covered the box is returned to the starting-point, and the operation is repeated with a grout somewhat thicker than the first. If this second application is not sufficient to fill the joints, the operation is repeated as often as may be necessary to fill them. If the grout has been made too thin, or the grade of the street is so great that the grout will not remain long enough in place to set, dry cement may be sprinkled over the joints and swept in. After the joints are completely filled and inspected, allowing three or four hours to intervene, the completed pavement should be covered with sand to a depth of about half an inch, and the roadway barricaded, and no traffic allowed on it for at least ten days.

The object of covering the pavement with sand is to prevent the grout from drying or settling too rapidly; hence, in dry and windy weather, it should be sprinkled from time to time. If coarse sand is employed in the grout, it will separate from the cement during the operation of filling the joints, with the result that many joints will be filled with sand and very little cement, while others will be filled with cement and little or no sand; thus there will be many spots in the pavement in which no bond is formed between the bricks, and under the action of traffic these portions will quickly become defective.

The coal-tar filler is best applied by pouring the material from buckets, and brooming it into the joints with wire brooms. In order to fill the joints effectually, it must be used only when very hot. To secure this condition, a heating tank on wheels is necessary. It should have a capacity of at least five barrels, and be kept at a uniform temperature all day. One man is necessary to feed the fire and draw the material into the buckets; another, to carry the buckets from the heating-tank to a third, who pours the material over the street. The latter starts to pour in the center of the street, working backward toward the curb, and pouring a strip about two feet in width. A fourth man, with a wire broom, follows immediately after him, sweeping the surplus material toward the pourer and in the direction of the curb. This method leaves the entire surface of the pavement covered with a thin coating of pitch, which should immediately be covered with a light coating of sand; the sand becomes imbedded in the pitch. Under the action of traffic, this thin coating is quickly worn away, leaving the surface of the bricks clean and smooth.

Tools Employed in Construction of Block Pavements. The principal tools required in constructing block pavements comprise *hammers* and *rammers* of varying sizes and shapes, depending on the material and size of the blocks to be laid; also *crowbars, sand screens,* and rattan and wire *brooms.* Cobblestones, square blocks, and brick require different types of both hammers and rammers for adjusting them to place and forcing them to their seat. A cobblestone rammer, for example, is usually made of wood (generally locust) in the shape of a long truncated cone, banded with iron at top and bottom, weighing about 40 pounds, and having two handles, one at the top and another on one side. A Belgian block rammer is slightly heavier, consisting of an upper part of wood set in a steel base; while a rammer for granite blocks is still heavier, comprising an iron base with cast-steel face, into which is set a locust plug with hickory handles. For laying brick, a wooden rammer shod with cast iron or steel and weighing about 27 pounds is used. A light rammer of about 20 pounds' weight, consisting of a metallic base attached to a long, slim wooden handle, is used for miscellaneous work, such as tamping in trenches, next to curbs, etc.

Concrete-Mixing Machine. Where large quantities of concrete are required, as in the foundations of improved pavements, concrete can be prepared more expeditiously and economically by the use of mechanical mixers, and the ingredients will be more thoroughly mixed, than by hand. Thorough incorporation of the ingredients is an essential element in the quality of a concrete. When mixed by hand, however, the incorporation is rarely complete, because it depends upon the proper manipulation of the hoe and shovel. The manipulation, although extremely simple, is rarely performed by the ordinary laborer unless he is constantly watched by the overseer.

Several varieties of concrete-mixing machines are in the market. A convenient portable type is illustrated in Fig. 60. The capacity of the mixers ranges from five to twenty cubic yards per hour, depending upon size, regularity with which the materials are supplied, speed, etc.

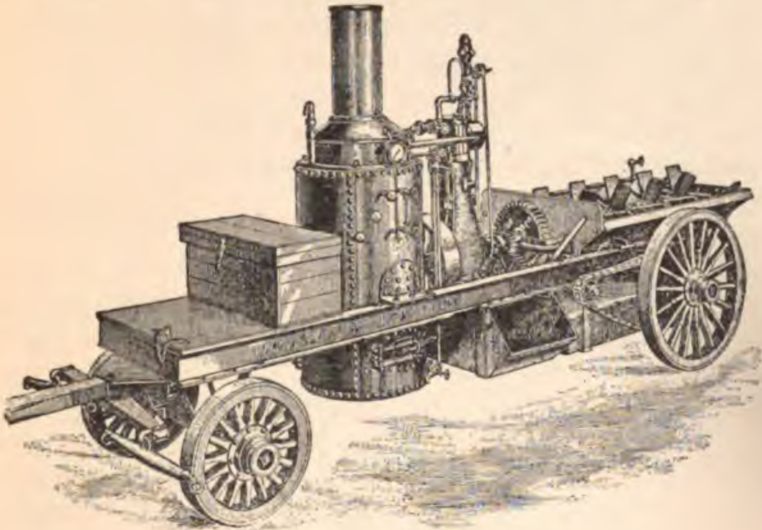


Fig. 60. Concrete Mixing Machine.

Gravel Heaters. Fig. 61 illustrates a device commonly employed for heating the gravel used for joint filling in stone-block pavements. These heaters are made in various sizes, a common size being 9 feet long, 5 feet wide, and 3 feet 9 inches high.

Melting Furnaces, for heating the pitch or tar for joint filling, are illustrated by Fig. 62. Various sizes are on the market.

WOOD PAVEMENTS

Wood pavements are formed of either rectangular or cylindrical blocks of wood. The rectangular blocks are generally 3 inches wi

9 inches long, and 6 inches deep; the round blocks are commonly 6 inches in diameter and 6 inches long.

The kinds of wood most commonly used are cedar, cypress, juniper, yellow pine, and mesquite; and recently *jarrah* from Australia, and *pyingado* from India, have been used.

The wood is used in its natural condition, or impregnated with creosote or other chemical preservative.

The blocks of wood are laid either on the natural soil, on a bed of sand and gravel, on a layer of broken stone, on a layer of concrete, or, sometimes, on a double layer of plank. The joints are filled either with sand, paving-pitch, or Portland-cement grout.

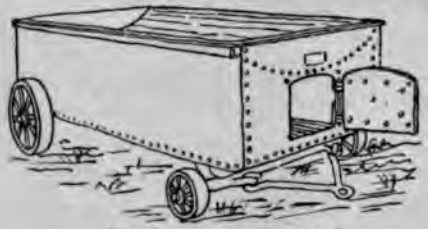


Fig. 61. Gravel Heater.

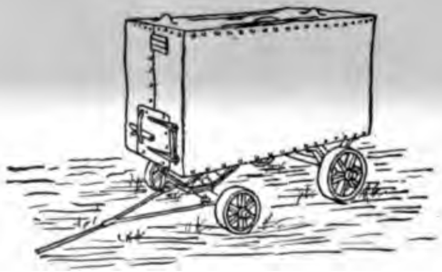


Fig. 62. Melting Furnace.

Advantages. The advantages of wood pavement may be stated as follows:

(1) It affords good foothold for horses.

(2) It offers less resistance to traction than stone, and slightly more than asphalt.

(3) It suits all classes of traffic.

(4) It may be used on grades up to five per cent.

(5) It is moderately durable.

(6) It yields no mud when laid upon an impervious foundation.

(7) It yields but little dust.

(8) It is moderate in first cost.

(9) It is not disagreeably noisy.

Defects. The principal objections to wood pavements are:

(1) It is difficult to cleanse.

(2) Under certain conditions of the atmosphere it becomes greasy and very unsafe for horses.

(3) It is not easy to open for the purpose of gaining access to underground pipes, it being necessary to remove rather a large surface for this purpose, which has to be left a little time after being repaired before traffic is again allowed upon it.

(4) It is absorbent of moisture.

(5) It is claimed by many that wood pavements are unhealthy.

Quality of Wood. The question as to which of the various kinds of wood available is the most durable and economical, has not been satisfactorily determined. Many varieties have been tried. In England, preference is given to Baltic fir, yellow pine, and Swedish yellow deal. In the United States the variety most used (on account of its abundance and cheapness) is cedar, but yellow pine, tamarack, and mesquite have also been used to a limited extent, and cypress and juniper are being largely used in some of the Southern States.

Hardwoods, such as oak, etc., do not make the best pavements, as such woods become slippery. The softer, close-grained woods, such as cedar and pine, wear better and give good foothold.

The wood employed should be sound and seasoned, free from sap, shakes, and knots. Defective blocks laid in the pavement will quickly cause holes in the surface, and the adjoining blocks will suffer under wear, the whole surface becoming bumpy.

Chemical Treatment of Wood. The great enemy of all wood pavements is decay, induced by the action of the air and water. Wood is porous, absorbs moisture, and thus hastens its own destruction. Many processes have been invented to overcome this defect. The most popular processes at present are *creosoting* and modifications of the same, known as the "creo-resinate" and "kreodine" processes. These consist of creosote mixed with various chemicals which are supposed to add to the preserving qualities of the creosote.

Creosoting. This process consists in impregnating the wood with the oil of tar, called *creosote*, from which the ammonia has been expelled, the effect being to coagulate the albumen and thereby prevent its decomposition, also to fill the pores of the wood with a bituminous substance which excludes both air and moisture, and which is noxious to the lower forms of animal and vegetable life. In adopting this process, all moisture should be dried out of the pores of the timber. The softer woods, while warm from the drying-house, may be immersed at once in an open tank containing hot creosote oil, when they will absorb about 8 or 9 pounds per cubic foot. For hardwoods, and woods which are required to absorb more than 8 or 9 pounds of creosote per cubic foot, the timber should be placed in an iron cylinder with closed ends, and the creosote, which should be heated to a temperature of about 120° F., forced in with a pressure of 170 pounds to the square inch. The heat must be kept up until the process is complete, to prevent the creosote from crystallizing in the pores of the wood. By this means the softer woods will easily absorb from 10 to 12 pounds of oil per cubic foot.

The most effective method, however, is to exhaust the air from the cylinder after the timber is inserted; then to allow the oil to flow in; and when the cylinder is full, to use a force pump with a pressure of 150 to 200 pounds per square inch, until the wood has absorbed the requisite quantity of oil, as indicated by a gauge, which should be fitted to the reservoir tank.

The oil is usually heated by coils of pipe placed in the reservoir, through which a current of steam is passed.

The quantity of creosote oil recommended to be forced into the wood is from 8 to 12 pounds per cubic foot. Into oak and other hard woods it is difficult to force, even with the greatest pressure, more than 2 or 3 pounds of oil.

The advantages of this process are: The chemical constituents of the oil preserve the fibers of the wood by coagulating the albumen of the sap; the fatty matters act mechanically by filling the pores and thus exclude water; while the carbolic acid contained in the oil is a powerful disinfectant.

The life of the wood is extended by any of the above processes, by preserving it from decay; but such processes have little or no effect on the wear of the blocks under traffic.

The process of dipping the blocks in coal tar or creosote oil is injurious. Besides affording a cover for the use of defective or sappy wood, it hastens decay, especially of green wood; it closes up the exterior of the cells of the wood so that moisture cannot escape, thus causing fermentation to take place in the interior of the block, which quickly destroys the strength of the fibers and reduces them to punk.

Expansion of Blocks. Wood blocks expand on exposure to moisture; and, when they are laid end to end across the street, the curbstones are liable to be displaced, or the courses of the blocks will be bent into reserve curves. To avoid this, the joints of the courses near the curb may be left open until expansion has ceased, the space being temporarily filled with sand. The rate of expansion is about 1 inch in 8 feet, but varies for different woods. The time required for the wood to become fully expanded varies from 12 to 18 months. By employing blocks impregnated with the oil of creosote, this trouble will be avoided. Blocks so treated do not contract or expand to any appreciable extent.

The comparative expansion of creosoted and plain wood blocks after immersion in water for forty-eight hours, in percentage on original dimensions, was:

Expansion of Wood Paving Blocks

DIMENSION	CREOSOTED	PLAIN
On length of block	.029	.6
On width " "	.57	.83
On depth " "	.15	.31

Manner of Laying. The blocks are set with the fiber vertical, and the long dimension crosswise of the street, the longitudinal joints being broken by a lap of at least one-third the length of the block; the blocks should be laid so as to have the least possible width of joint. Wide joints hasten the destruction of the wood by permitting the fibers to wear under traffic, which also causes the surface of the pavement

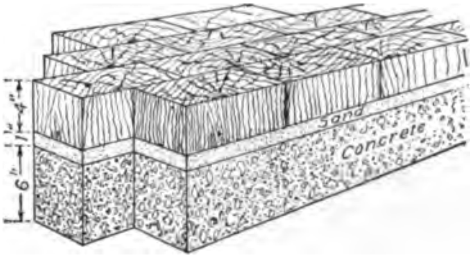


Fig. 63.

to wear in small ridges. The most recent practice for laying blocks on 3 per cent grades, has been to remove from the top of one side of each block a strip ¼ inch thick and 1½ inches deep, extending the length of the block. When the blocks are laid and driven closely together, there is a quarter-inch opening or joint extending clear across the street in each course. These joints are filled with Portland cement grout. Fig. 63 shows a section of pavement having this form of joint.

Filling for Joints. The best materials for filling the joints are bitumen for the lower two or three inches, and hydraulic cement grout for the remainder of the depth. The cement grout protects the pitch from the action of the sun, and does not wear down very much below the surface of the wood.

ASPHALT PAVEMENTS.

Asphaltic Paving Materials. All asphaltic or bituminous pavements are composed of two essential parts—namely, the cementing material (*matrix*) and the resisting material (*aggregate*). Each has a distinct function to perform; the first furnishes and preserves the coherency of the mass; the second resists the wear of traffic.

Two classes of asphaltic paving compounds are in use,—namely, *natural* and *artificial*. The "natural" variety is composed of either limestone or sandstone naturally cemented with bitumen. To this class belong the bituminous limestones of Europe, Texas, Utah, etc., and the bituminous sandstones of California, Kentucky, Texas, Indian Territory, etc. The "artificial" consists of mixtures of asphaltic cement with sand and stone dust. To this class belong the pavements made from Trinidad, Bermudez, Cuban, and similar asphaltums. For the artificial variety, most hard bitumens are, when properly prepared, equally suitable. For the aggregate, the most suitable materials are stone-dust from the harder rocks, such as granite, trap, etc., and sharp angular sand. These materials should be entirely free from loam and vegetable impurities. The strength and enduring qualities of the mixture will depend upon the quality, strength, and proportion of each ingredient, as well as upon the cohesion of the matrix and its adhesion to the aggregate.

Bituminous limestone consists of carbonate of lime naturally cemented with bitumen in proportions varying from 80 to 93 per cent of carbonate of lime and from 7 to 20 per cent of bitumen. Its color, when freshly broken, is a dark (almost black) chocolate brown, the darker color being due to a large percentage of bitumen. At a temperature of from 55° to 70° F., the material is hard and sonorous, and breaks easily with an irregular fracture; at temperatures between 70° and 140° F. it softens, passing with the rise in temperature through various degrees of plasticity, until, at between 140° and 160° F., it begins to crumble; at 212° it commences to melt; and at 280° F. it is completely disintegrated. Its specific gravity is about 2.235.

Bituminous limestone is the material employed for paving purposes throughout Europe. It is obtained principally from deposits at Val-de-Travers, canton of Neufchatel, Switzerland; at Seysell, in the Department of Ain, France; at Ragusa, Sicily; at Limmer, near Hanover; and at Vorwohle, Germany.

Bituminous limestone is found in several parts of the United States. Two of these deposits are at present being worked—one in Texas, the material from which is called "lithocarbon"; and one on the Wasatch Indian Reservation. These deposits contain from 10 to 30 per cent of bitumen.

The bituminous limestones which contain about 10 per cent of bitumen are used for paving in their natural condition, being simply reduced to powder, heated until thoroughly softened, then spread while hot upon the foundation, and tamped and rammed until compacted.

Bituminous sandstones are composed of sandstone rock impregnated with bitumen in amounts varying from a trace to 70 per cent. They are found in both Europe and America. In Europe, they are chiefly used for the production of pure bitumen, which is extracted by boiling or macerating them with water. In the United States, extensive deposits are found in the Western States; and since 1880 they have been gradually coming into use as a paving material, so that now upwards of 150 miles of streets in Western cities are paved with them. They are prepared for use as paving material by crushing to powder, which is heated to about 250° F. or until it becomes plastic, then spread upon the street and compressed by rolling; sometimes sand or gravel is added, and it is stated that a mixture of about 80 per cent of gravel makes a durable pavement.

Trinidad Asphaltum. The deposits of asphaltum in the island of Trinidad, W. I., have been the main source of supply for the asphaltum used in street paving in the United States. Three kinds are found there, which have been named, according to the source, *lake-pitch*, *land* or *overflow pitch*, and *iron pitch*. The first and most valuable kind is obtained from the so-called Pitch Lake.

The term *land* or *overflow* pitch is applied to the deposits of asphaltum found outside the lake. These deposits form extensive beds of variable thickness, and are covered with from a few to several feet of earth; they are considered by some authorities to be formed from pitch which has overflowed from the lake; by others to be of entirely different origin. The name *cheese pitch* is given to such portions of the land pitch as more nearly resemble that obtained from the lake.

The term *iron pitch* is used to designate large and isolated masses of extremely hard asphaltum found both within and without the borders of the lake. It is supposed to have been formed by the action of heat caused by forest fires, which, sweeping over the softer pitch, removed its more volatile constituents.

The name *épurée* is given to asphaltum refined on the island of Trinidad. The process is conducted in a very crude manner, in large, open, cast-iron sugar boilers.

The characteristics of crude Trinidad asphaltum, both *lake* and *land*, are as follows: It is composed of bitumen mixed with fine sand, clay, and vegetable matter. Its specific gravity varies according to the impurities present, but is usually about 1.28. Its color, when freshly excavated, is a brown, which changes to black on exposure to the atmosphere. When freshly broken, it emits the usual bituminous odor. It is porous, containing gas cavities, and in consistency resembles cheese. If left long enough in the sun, the surface will soften and melt, and will finally flow into a more or less compact mass.

Refined Trinidad Asphaltum. The crude asphaltum is refined or purified by melting it in iron kettles or stills by the application of indirect heat.

The operation of refining proceeds as follows: During the heating, the water and lighter oils are evaporated; the asphaltum is liquefied; the vegetable matter rises to the surface, and is skimmed off; the earthy and siliceous matters settle to the bottom; and the liquid asphaltum is drawn off into old cement or flour barrels.

When the asphaltum is refined without agitation, the residue remaining in the still forms a considerable percentage of the crude material, frequently amounting to 12 per cent; and it was at one time considered that the greater the amount of this residue the better the quality of the refined asphaltum. Since agitation has been adopted, however, the greater part of the earthy and siliceous matters is retained in suspension; and it has come to be considered just as desirable for a part of the surface mixture as the sand which is subsequently added. The refined asphaltum, if for local use, is generally converted into cement in the same still in which it was refined.

The average composition of both the land and lake varieties is shown by the following analyses:

Average Composition of Trinidad Asphaltum

CONSTITUENTS	LAKE		LAND
	Hard	*Soft*	
	Per Cent	Per Cent	Per Cent
Water	27.85	34.10	26.62
Inorganic matter	26.38	25.05	27.57
Organic non-bituminous matter	7.63	6.35	8.05
Bitumen	38.14	34.50	37.76
	100.00	100.00	100.00
When the analyses are calculated to a basis of dry substances, the composition is: Inorganic matter	36.56	38.00	37.74
Organic matter not bitumen	10.57	9.64	10.68
Bitumen	52.87	52.36	51.58
	100.00	100.00	100.00
The substances volatilized in 10 hours at 400° F	3.66	12.24	0.86 to 1.37
The substances soften at	190° F.	170° F.	200° to 250° F.
" " flow at	200° F.	185° F.	210° to 328° F.

The characteristics of refined Trinidad asphaltum are as follows: The color is black, with a homogeneous appearance. At a temperature of about 70° F., it is very brittle, and breaks with a conchoidal fracture. It burns with a yellowish-white flame, and in burning emits an empyreumatic odor, and possesses little cementitious quality. To give it the required plasticity and tenacity, it is mixed while liquid with from 16 to 21 pounds of residuum oil to 100 pounds of asphaltum.

The product resulting from the combination is called *asphalt paving-cement.* Its consistency should be such that, at a temperature of from 70° to 80° F., it can be easily indented with the fingers, and on slight warming be drawn out in strings or threads.

Artificial Asphalt Pavements. The pavements made from Trinidad, Bermudez, California, and similar asphaltums, are composed of mechanical mixtures of asphaltic cement, sand, and stone-dust.

The sand should be equal in quality to that used for hydraulic cement mortar; it must be entirely free from clay, loam, and vegetable impurities; its grains should be angular and range from coarse to fine.

The stone-dust is used to aid in filling the voids in the sand and thus reduce the amount of cement. The amount used varies with the coarseness of the sand and the quality of the cement, and ranges from 5 to 15 per cent. (The voids in sand vary from .3 to .5 per cent.)

As to the quality of the stone-dust, that from any durable stone is equally suitable. Limestone-dust was originally used, and has never been entirely discarded.

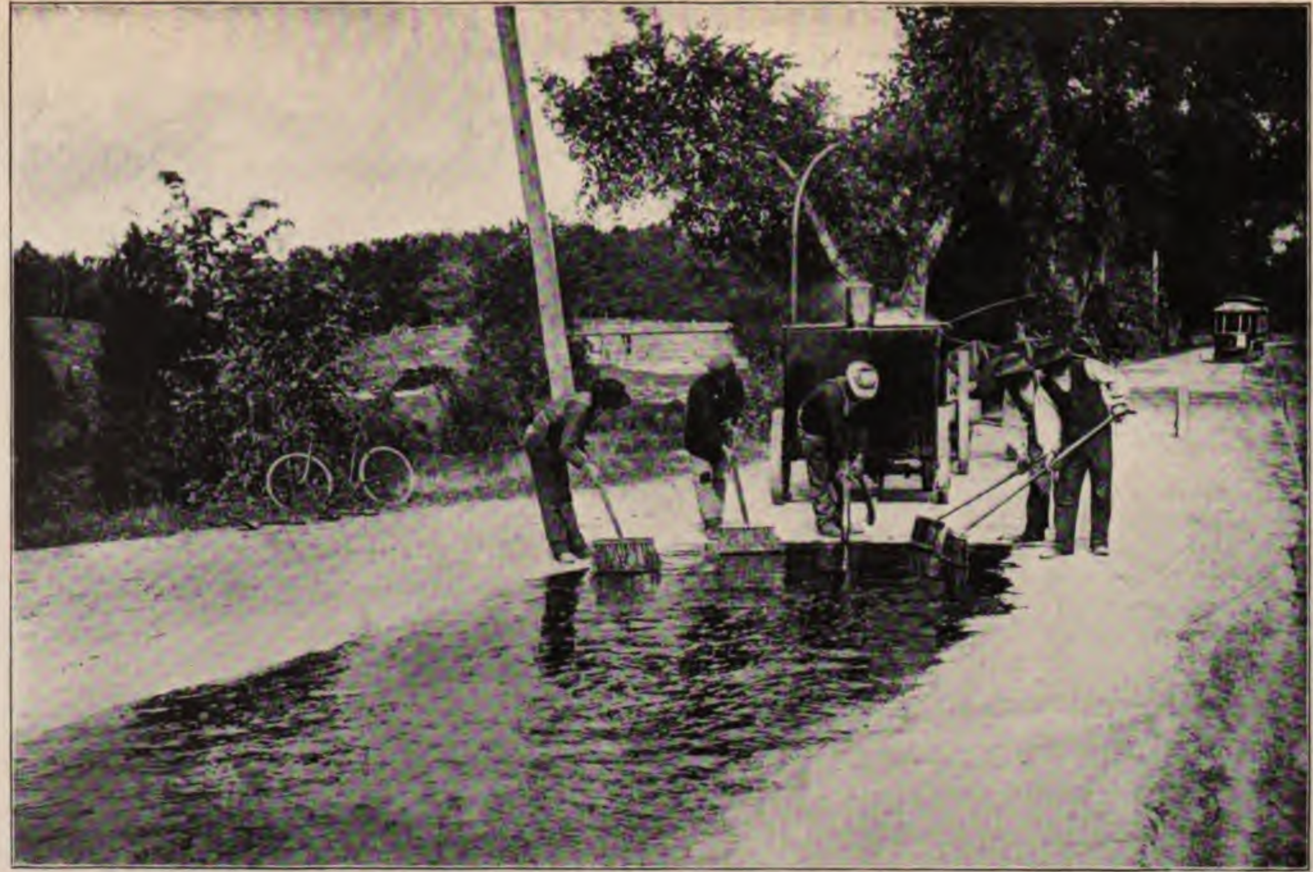
The paving composition is prepared by heating the mixed sand and stone-dust and the asphalt cement separately to a temperature of about 300° F. The heated ingredients are measured into a pug-mill and thoroughly incorporated. When this is accomplished, the mixture is ready for use. It is hauled to the street and spread with iron rakes to such depth as will give the required thickness when compacted (the finished thickness varies between 1½ and 2½ inches). The reduction of thickness by compression is generally about 40 per cent.

The mixture is sometimes laid in two layers. The first is called the **binder** or **cushion-coat**; it contains from 2 to 5 per cent more cement than the surface-coat; its thickness is usually ½ inch. The object of the binder course is to unite the surface mixture with the foundation, which it does through the larger percentage of cement that it contains, which, if put in the surface mixture, would render it too soft.

The paving composition is compressed by means of rollers and tamping irons, the latter being heated in a fire contained in an iron basket mounted on wheels. These irons are used for tamping such portions as are inaccessible to the roller—namely, gutters, around manhole heads, etc.

Two rollers are sometimes employed; one, weighing 5 to 6 tons and of narrow tread, is used to give the first compression; and the other, weighing about 10 tons and of broad tread, is used for finishing. The amount of rolling varies; the average is about 1 hour per 1,000 square yards of surface. After the primary compression, natural hydraulic or any impalpable mineral matter is sprinkled over the surface, to prevent the adhesion of the material to the roller and to give the surface a more pleasing appearance. When the asphalt is laid up to the curb, the surface of the portion forming the gutter is painted with a coat of hot cement.

Although asphaltum is a bad conductor of heat, and the cement



STATE HIGHWAY AT WESTWOOD, MASSACHUSETTS

Showing the application of hot coal tar to the surface of a macadam road, sometimes called "tar painting." The road is first thoroughly cleaned; and the tar, heated to about 180° F., is run onto the road and distributed by hand brooms. About half a gallon of tar to the square yard is used.

Courtesy of Massachusetts Highway Commission.

retains its plasticity for several hours, occasions may and do arise through which the composition before it is spread has cooled; its condition when this happens is analogous to hydraulic cement which has taken a "set," and the same rules which apply to hydraulic cement in this condition should be respected in regard to asphaltic cement.

The proportions of the ingredients in the paving mixture are not constant, but vary with the climate of the place where the pavement is to be used, the character of the sand, and the amount and character of the traffic that will use the pavement. The range in the proportion is as follows:

Formula for Asphaltic Paving Mixture

Asphalt cement	12 to 15 per cent.
Sand	70 to 83 " "
Stone-dust	5 to 15 " "

A cubic yard of the prepared material weighs about 4,500 pounds, and will lay the following amount of wearing-surface:

2½ inches thick	12 square yards.
2 " "	18 " "
1½ " "	27 " "

One ton of refined asphaltum makes about 2,300 pounds of asphalt cement, equal to about 3.4 cubic yards of surface material.

Foundation. A solid, unyielding foundation is indispensable with all asphaltic pavements, because asphalt of itself has no power of offering resistance to the action of traffic, consequently it is nearly always placed upon a bed of hydraulic cement concrete. The concrete must be thoroughly set and its surface dry before the asphalt is laid upon it; if not, the water will be sucked up and converted into steam, with the result that coherence of the asphaltic mixture is prevented, and, although its surface may be smooth, the mass is really honeycombed, so that as soon as the pavement is subjected to the action of traffic, the voids or fissures formed by the steam appear on the surface, and the whole pavement is quickly broken up.

Advantages of Asphalt Pavement. These may be summed up as follows:

(1) Ease of traction.
(2) It is comparatively noiseless under traffic.
(3) It is impervious.
(4) It is easily cleansed.
(5) It produces neither mud nor dust.
(6) It is pleasing to the eye.

(7) It suits all classes of traffic.

(8) There is neither vibration nor concussion in traveling over it.

(9) It is expeditiously laid, thereby causing little inconvenience to traffic.

(10) Openings to gain access to underground pipes are easily made.

(11) It is durable.

(12) It is easily repaired.

Defects of Asphalt Pavement. These are as follows:

(1) It is slippery under certain conditions of the atmosphere. The American asphalts are much less so than the European, on account of their granular texture derived from the sand. The difference is very noticeable; the European are as smooth as glass, while the American resemble fine sandpaper.

(2) It will not stand constant moisture, and will disintegrate if excessively sprinkled.

(3) Under extreme heat it is liable to become so soft that it will roll or creep under traffic and present a wavy surface; and under extreme cold there is danger that the surface will crack and become friable.

(4) It is not adapted to grades steeper than 2½ per cent, although it is in use on grades up to 7.30 per cent.

(5) Repairs must be quickly made, for the material has little coherence, and if, from irregular settlement of foundation or local violence, a break occurs, the passing wheels rapidly shear off the sides of the hole, and it soon assumes formidable dimensions.

The strewing of sand upon asphalt renders it less slippery; but in addition to the interference of the traffic while this is being done, there are further objections—namely, the possible injury by the sand cutting into the asphalt, the expense of labor and materials, and the mud formed, which has afterwards to be removed.

Although pure asphaltum is absolutely impervious and insoluble in either fresh or salt water, yet asphalt pavements in the continued presence of water are quickly disintegrated. Ordinary rain or daily sprinkling does not injure them when they are allowed to become perfectly dry again. The damage is most apparent in gutters and adjacent to overflowing drinking fountains. This defect has long been recognized; and various measures have been taken to overcome it, or at least to reduce it to a minimum. In some cities, ordinances have been passed, seeking to regulate the sprinkling of the streets; and in many places the gutters are laid with stone or vitrified brick (see Figs. 64 and 65), while in others the asphalt is laid to the curb, a space of 12 to 15 inches along the curb being covered with a thin coating of asphalt cement.

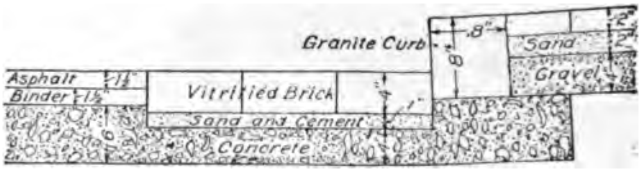


Fig. 64.

Asphalt laid adjoining center-bearing street-car rails is quickly broken down and destroyed. This defect is not peculiar to asphalt. All other materials when placed in similar positions are quickly worn. Granite blocks laid along such tracks have been cut into at a rate of more than half an inch a year. The frequent entering and turning off of vehicles from car tracks is one of the severest tests that can be applied to any paving material; moreover, the gauge of trucks and vehicles is frequently greater than that of the rails, so one wheel runs on the rail and the other outside. The number of wheels thus traveling in one line must quickly wear a rut in any material adjoining the center-bearing rail.

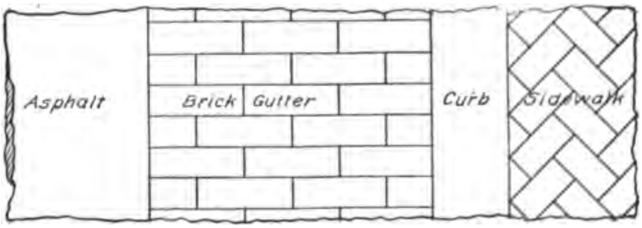


Fig. 65.

To obviate the destruction of asphalt in such situations, it is usual to lay a strip of granite block or brick paving along the rail. This pavement should be of sufficient width to support the wheels of the widest gauge using the street.

The burning of leaves or making of fires on asphalt pavements should not be permitted, as it injures the asphalt, and the paving companies cannot be compelled to repair the damaged places without compensation.

Asphalt Blocks. Asphalt paving blocks are formed from a mixture of asphaltic cement and crushed stone in the proportion of 8 to 12 per cent of cement to 88 and 92 per cent of stone. The materials are heated to a temperature of about 300° F., and mixed while hot in a suitable vessel. When the mixing is complete, the material is placed in moulds and subjected to heavy pressure, after which the blocks are cooled suddenly by plunging into cold water.

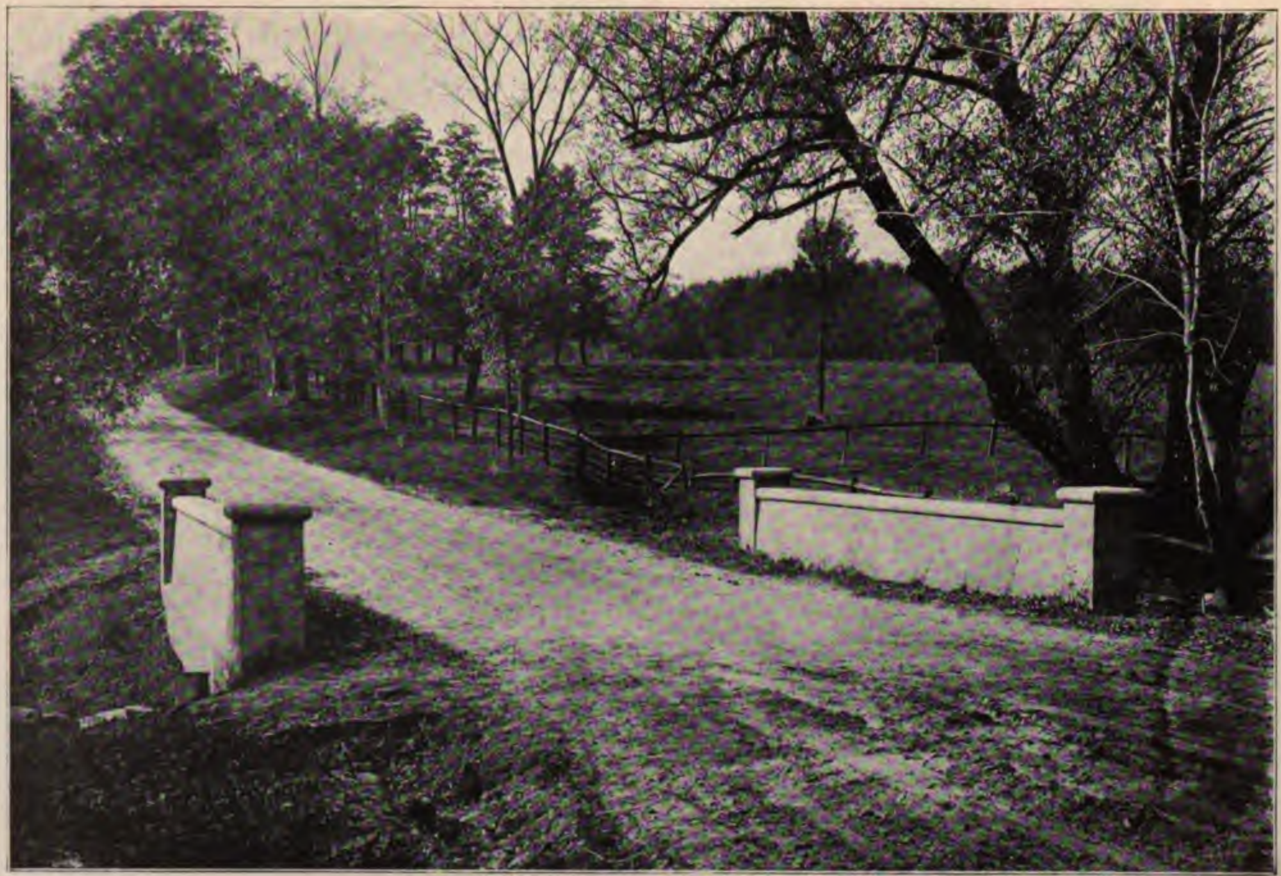
The usual dimensions of the blocks are 4 inches wide, 3 inches deep, and 12 inches long.

Foundation. The blocks are usually laid upon a concrete foundation with a cushion-coat of sand about ½ inch thick. They are laid with their length at right angles to the axis of the street, and the longitudinal joints should be broken by a lap of at least 4 inches. The blocks are then either rammed with hand rammers or rolled with a light steam roller, the surface being covered with clean, fine sand; no joint filling is used, as, under the action of the sun and traffic, the blocks soon become cemented.

The advantages claimed for a pavement of asphalt blocks over a continuous sheet of asphalt are: (1) That they can be made at a factory located near the materials, whence they can be transported to the place where they are to be used and can be laid by ordinary paviors, whereas sheet pavements require special machinery and skilled labor; (2) that they are less slippery, owing to the joints and the rougher surface due to the use of crushed stone.

Asphalt Macadam—Bituminous Macadam. Recently it has been proposed to use asphalt as a binding material for broken stone. There are two patented processes—the Whinery and the Warren—which differ slightly in details.

The advantages claimed for these methods are: (1) The first coat will be materially less; (2) it will offer a better foothold for horses; (3) it will be at least as durable as the ordinary sheet asphalt; (4) it will not shift under traffic and roll into waves; (5) it will not crack; (6) it



CULVERT AT LENOX, MASSACHUSETTS

Showing concrete parapet walls. The effect is more pleasing than that produced by the more usual pipe rail fence.

Courtesy of Massachusetts Highway Commission.

can be repaired more cheaply and with less skilled labor than can the ordinary sheet asphalt.

Tools Employed in Construction of Asphalt Pavements. The tools used in laying sheet asphalt pavements comprise iron rakes; hand rammers; smoothing irons (Fig. 67); pouring pots (Fig. 69);

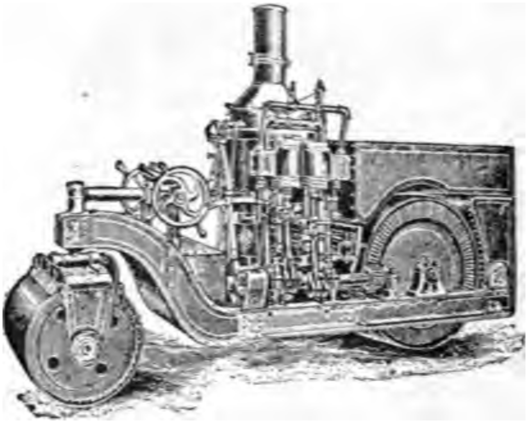


Fig. 66. Steam Roller.

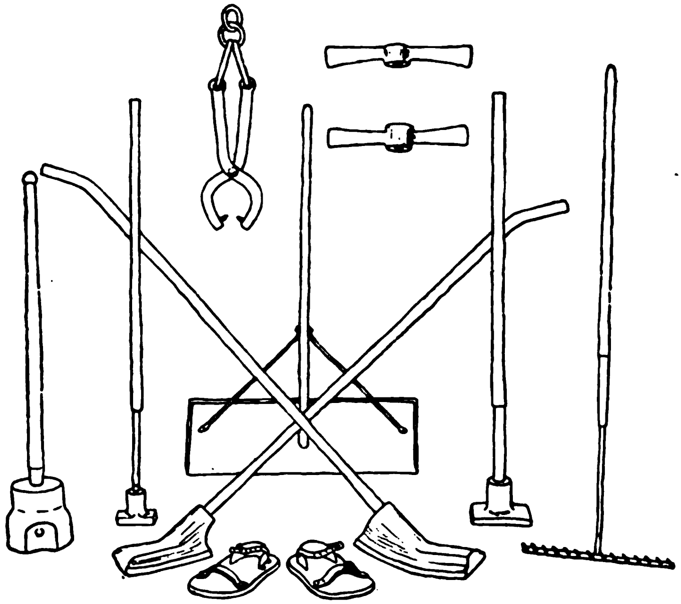


Fig. 67. Asphalt Tools.

hand rollers, either with or without a fire-pot (Fig. 68); and steam rollers, with or without provision for heating the front roll (Fig. 66). These rollers are different in construction, appearance, and weight

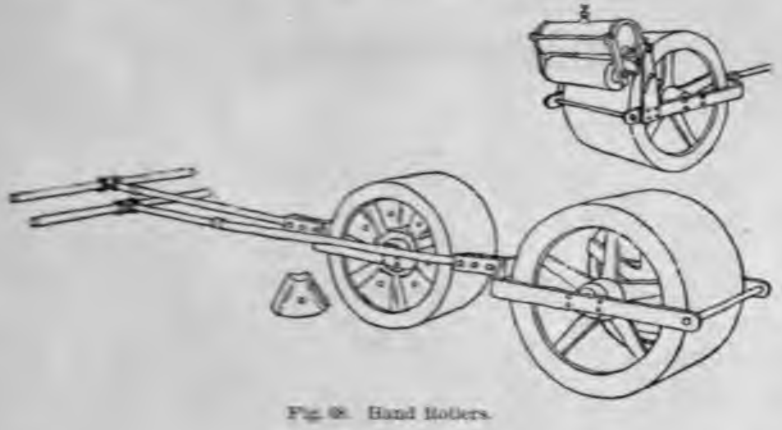
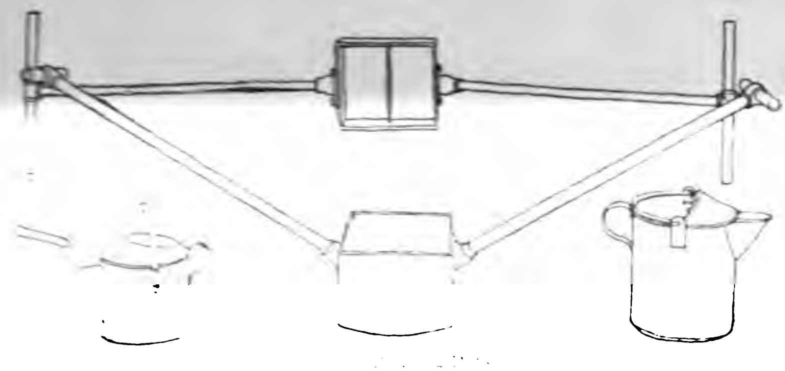


Fig. 68. Hand Rollers.



[illegible]

[illegible] compacting broken stone. The difference [illegible] of the work required.

[illegible] of a five-ton roller are as follows:

[illegible]	30 to 32 inches diameter.
[illegible]	18 " "
[illegible]	40 " "
[illegible]	[illegible] " "
[illegible]	14 feet.
[illegible]	7 to 8 feet.
[illegible]	80 to 100 gallons.
[illegible]	200 pounds.

FOOTPATHS—CURBS—GUTTERS.

A footpath or walk is simply a road under another name—a road for pedestrians instead of one for horses and vehicles. The only difference that exists is in the degree of service required; but the conditions of consruction that render a road well adapted to its object are very much the same as those required for a walk.

The effects of heavy loads such as use carriageways are not felt upon footpaths; but the destructive action of water and frost is the same in either case, and the treatment to counteract or resist these elements as far as practicable, and to produce permanency, must be the controlling idea in each case, and should be carried out upon a common principle. It is not less essential that a walk should be well adapted to its object than that a road should be; and it is annoying to find it impassable or insecure and in want of repair when it is needed for convenience or pleasure. In point of economy, there is the same advantage in constructing a footway skilfully and durably as there is in the case of a road.

Width. The width of footwalks (exclusive of the space occupied by projections and shade trees) should be ample to accomodate comfortably the number of people using them. In streets devoted entirely to commercial purposes, the clear width should be at least one-third the width of the carriageway; in residential and suburban streets, a very pleasing result can be obtained by making the walk one-half the width of the roadway, and devoting the greater part to grass and shade trees.

Cross Slope. The surface of footpaths must be sloped so that the surface water will readily flow to the gutters. This slope need not be very great; $\frac{1}{8}$ inch per foot will be sufficient. A greater slope with a thin coating of ice upon it, becomes dangerous to pedestrians.

Foundation. As in the case of roadways, so with footpaths, the foundation is of primary importance. Whatever material may be used for the surface, if the foundation is weak and yielding, the surface will settle irregularly and become extremely objectionable, if not dangerous, to pedestrians.

Surface. The requirements of a good covering for sidewalks are:

(1) It must be smooth but not slippery.

(2) It must absorb the minimum amount of water, so that it may dry rapidly after rain.

(3) It must not be easily abraded.

(4) It must be of uniform quality throughout, so that it may wear evenly.

(5) It must neither scale nor flake.

(6) Its texture must be such that dust will not adhere to it.

(7) It must be durable.

Materials. The materials used for footpaths are as follows: Stone, natural and artificial; wood; asphalt; brick; tar concrete; and gravel.

Of the natural stones, sandstone (bluestone) and granite are extensively employed.

The bluestone, when well laid, forms an excellent paving material. It is of compact texture, absorbs water to a very limited extent, and hence soon dries after rain; it has sufficient hardness to resist abrasion, and wears well without becoming excessively slippery.

Granite, although exceedingly durable, wears very slippery, and its surface has to be frequently roughened.

Slabs, of whatever stone, must be of equal thickness throughout their entire area; the edges must be dressed true to the square for the whole thickness (edges must not be left feathered as shown in Fig. 70); and the slabs must be solidly bedded on the foundation and the joints filled with cement-mortar.

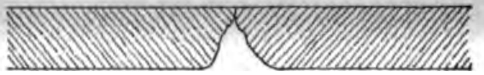


Fig. 70.

Badly set or faultily dressed flagstones are very unpleasant to walk over, especially in rainy weather; the unevenness causes pedestrians to stumble, and rocking stones squirt dirty water over their clothes.

Wood has been largely used in the form of planks; it is cheap in first cost, but proves very expensive from the fact that it lasts but a comparatively short time and requires constant repair to keep it from becoming dangerous.

Asphalt forms an excellent footway pavement; it is durable and does not wear slippery.

Brick. Brick of suitable quality, well and carefully laid on a concrete foundation, makes an excellent footway pavement for residential and suburban streets of large cities, and also for the main streets of smaller towns. The bricks should be a good quality of paving brick (ordinary building brick are unsuitable, as they soon wear out and are easily broken). The bricks should be laid in parallel rows on their edges, with their length at right angles to the axis of the path.

Curbstones. Curbstones are employed for the outer side of footways, to sustain the coverings and form the gutter. Their upper edges are set flush with the footwalk pavement, so that the water can flow over them into the gutters.

The disturbing forces which the curb has to resist, are: (1) The pressure of the earth behind it, which is frequently augmented by piles of merchandise, building materials, etc. This pressure tends to overturn it, break it transversely, or move it bodily on its base. (2) The pressure due to the expansion of freezing earth behind and beneath it. This force is most frequent where the sidewalk is partly sodded and the ground is accordingly moist. Successive freezing and thawing of the earth behind the curb will occasion a succession of thrusts forward, which, if the curb be of faulty design, will cause it to incline several degrees from the vertical. (3) The concussions and abrasions caused by traffic To withstand the destructive effect of wheels, curbs are faced with iron; and a concrete curb with a rounded edge of steel has been patented and used to some extent. Fires built in the gutters deface and seriously injure the curb. Posts and trees set too near the curb, tend to break, displace, and destroy it.

The use of drain tiles under the curb is a subject of much difference of opinion among engineers. Where the subsoil contains water naturally, or is likely to receive it from outside the curb-lines, the use of drains is of decided benefit; but great care must be exercised in jointing the drain-tiles, lest the soil shall be loosened and removed, causing the curb to drop out of alignment.

The materials employed for curbing are the natural stones, as granite, sandstone (bluestone), etc., artificial stone, fire-clay, and cast iron.

The dimensions of curbstones vary considerably in different localities and according to the width of the footpaths; the wider the path, the wider should be the curb. It should, however, never be less than 8 inches deep, nor narrower than 4 inches. Depth is necessary to prevent the curb turning over toward the gutter. It should never be in smaller lengths than 3 feet. The top surface should be beveled off to conform to the slope of the footpath. The front face should be hammer-dressed for a depth of about 6 inches, in order that there may be a smooth surface visible against the gutter. The back for 3 inches from the top should also be dressed, so that the flagging or other paving may butt fair against it. The end joints should be cut truly square, the full thickness of the stone at the top, and so much below the top as will be exposed, the remaining portion of the depth and bottom should be roughly squared, and the bottom should be fairly parallel to the top. (See Figs. 71 and 72).

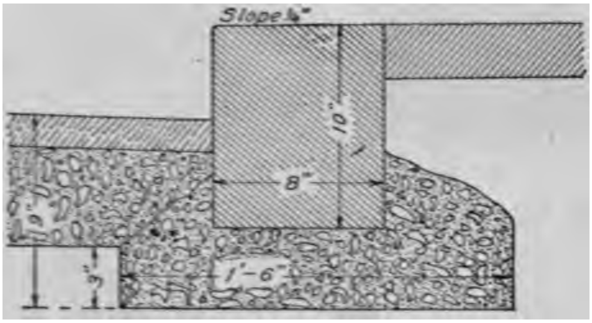


Fig. 71.

Artificial Stone. Artificial stone is being extensively used as a footway paving material. Its manufacture is the subject of several patents, and numerous kinds are to be had in the market. When manufactured of first-class materials and laid in a substantial manner, with proper provision against the action of frost, artificial stone forms a durable, agreeable, and inexpensive pavement.

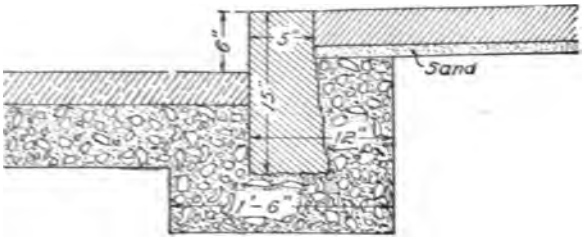


Fig. 72.

The varieties most extensively used in the United States are known by the names of *granolithic*, *monolithic*, *ferrolithic*, *kosmocrete*, *metalithic*, etc.

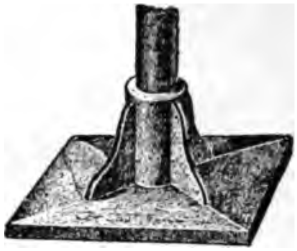


Fig. 73. Tamper.

The process of manufacture is practically the same for all kinds, the difference being in the materials employed. The usual ingredients are Portland cement, sand, gravel, and crushed stone.

Artificial stone for footway pavements is formed in two ways—namely, in blocks manufactured at a factory, brought on the ground, and laid in the same manner as natural stone; or the raw materials are brought upon the work, prepared, and laid in place, blocks being formed by the use of board moulds.

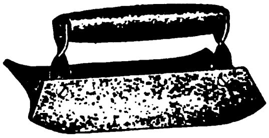


Fig. 74. Quarter-Round.

The manner of laying is practically the same for all kinds. The area to be paved is excavated to a minimum depth of 8 inches, and to such greater depths as the nature of the ground may require to secure a solid foundation. The surface of the ground so exposed is well compacted by ramming; and a layer of gravel, ashes, clinker, or other suitable material is spread and consolidated; on this is placed the concrete wearing surface, usually 4 inches thick. As a protection against the lifting effects of frost, the concrete is laid in squares, rectangles, or other forms having areas ranging from 6 to 30 square feet, strips of wood being employed to form moulds in which the concrete is placed. After the concrete is set, these strips are removed, leaving joints about half an inch wide between the blocks. Under some patents these joints are filled with cement; under others, with tarred paper; and in some cases they are left open.

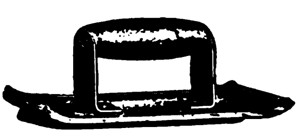


Fig. 75. Jointer.

Tools Employed in Construction of Artificial Stone Pavements. Tampers (Fig. 73). Cast iron, with hickory handle; range from 6 by 8 inches to 8 by 10 inches.

Quarter-Round, (Fig. 74). Made of any desired radius. Used for forming corners and edges.

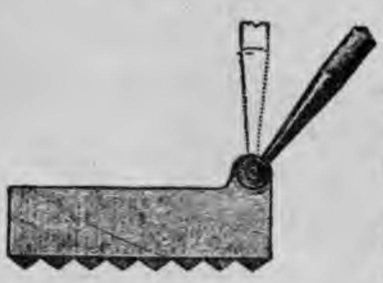


Fig. 76. Cutter.

Jointer (Fig. 75). Used for trimming and finishing the joints.

Cutter (Fig. 76). Used for cutting the concrete into blocks.

Gutter Tool (Fig. 77). Used for forming and finishing gutters.

Imprint Rollers (Figs. 78 and 79). Here are shown two designs of rollers for imprinting the surface of artificial stone pavements with grooves, etc.

SELECTING THE PAVEMENT

The problem of selecting the best pavement for any particular case is a local one, not only for each city, but also for each of the various parts into which the city is imperceptibly divided; and it involves so many elements that the nicest balancing of the relative values for each kind of pavement is required, to arrive at a correct conclusion.

In some localities, the proximity of one or more paving materials determines the character of the pavement; while in other cases a careful investigation may be required in order to select the most suitable material. Local conditions should always be considered; hence it is not possible to lay down any fixed rule as to what material makes the best pavement.

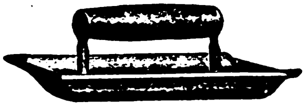


Fig. 77. Gutter Tool.

The qualities essential to a good pavement may be stated as follows:

(1) It should be impervious.

(2) It should afford good foothold for horses.

(3) It should be hard and durable, so as to resist wear and disintegration.

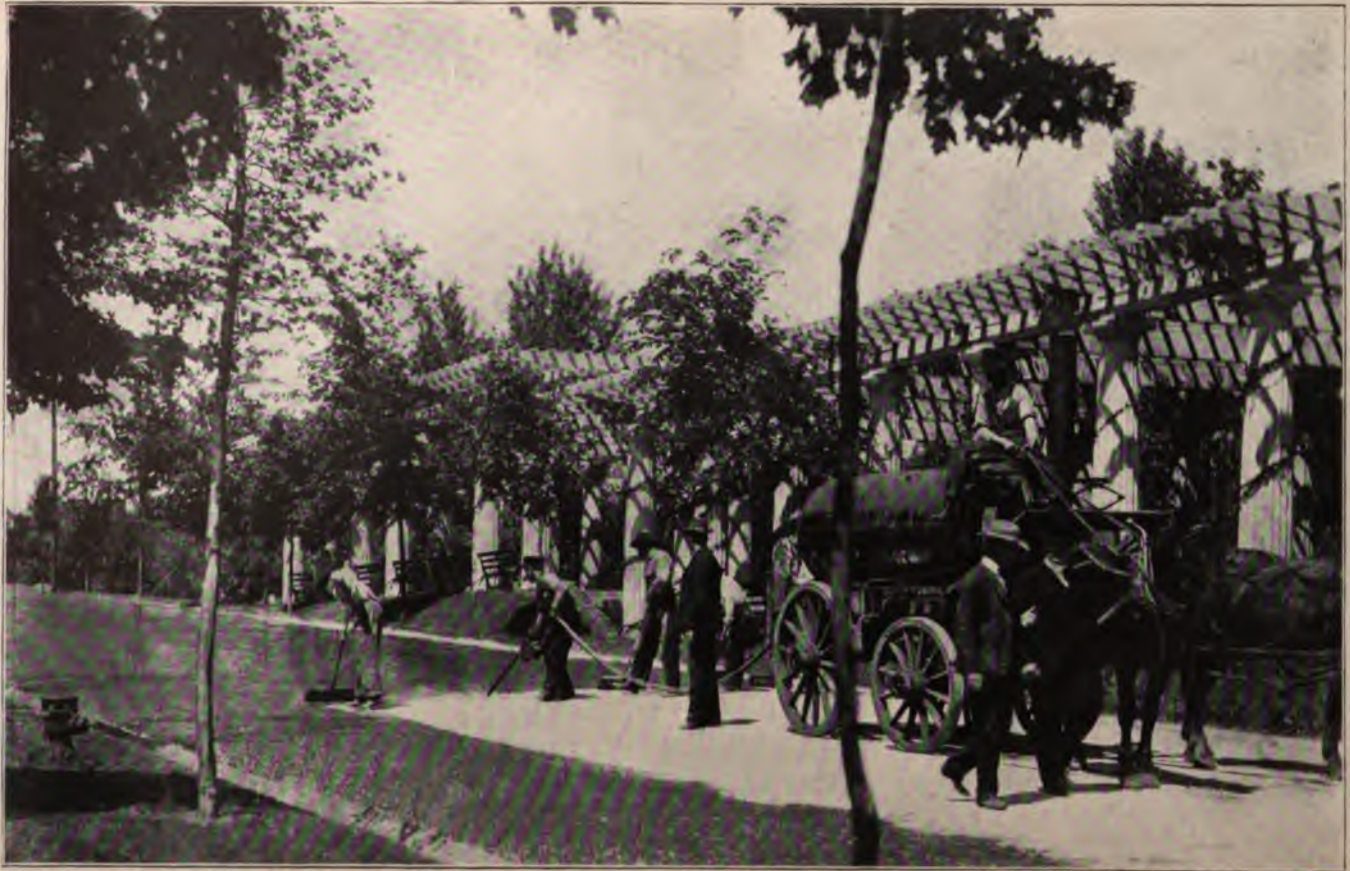
(4) It should be adapted to every grade.

(5) It should suit every class of traffic.

(6) It should offer the minimum resistance to traction.

(7) It should be noiseless.

(8) It should yield neither dust nor mud.



TREATING A ROADBED WITH A PATENT COMPOSITION TO PREVENT DUST AND PRESERVE THE SURFACE

View in Kansas City, Missouri.

(9) It should be easily cleaned.

(10) It should be cheap.

Interests Affected in Selection. Of the above requirements, numbers 2, 4, 5, and 6 affect the traffic and determine the cost of haulage by the limitations of loads, speed, and wear and tear of horses and vehicles. If the surface is rough or the foothold bad, the weight of the load a horse can draw is decreased, thus necessitating the making of more trips or the employment of more horses and vehicles to move a given weight. A defective surface necessitates a reduction in the speed of movement and consequent loss of time; it increases the wear of horses, thus decreasing their life service and lessening the value of their current services; it also increases the cost of maintaining vehicles and harness.

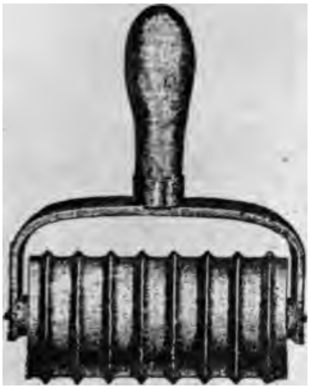


Fig. 78. Imprint Roller.

Numbers 7, 8, and 9 affect the occupiers of adjacent premises, who suffer from the effect of dust and noise; they also affect the owners of said premises, whose income from rents is diminished where these disadvantages exist. Numbers 3 and 10 affect the taxpayers alone—first, as to the length of time during which the covering remains serviceable; and second, as to the amount of the annual repairs. Number 1 affects the adjacent occupiers principally on hygienic grounds. Numbers 7 and 8 affect both traffic and occupiers.

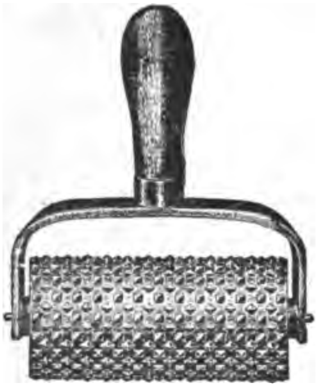


Fig. 79. Imprint Roller.

Problem Involved in Selection. The problem involved in the selection of the most suitable pavement consists of the following factors: (1) adaptability; (2) desirability; (3) serviceability; (4) durability; (5) cost.

Adaptability. The best pavement for any given roadway will depend altogether on local circumstances. Pavements must be adapted to the class of traffic that will use them. The pavement suitable for a road through an agricultural district will not be suitable for the streets of a manufacturing center; nor will the covering suitable for heavy traffic be suitable for a pleasure drive or for a residential district.

General experience indicates the relative fitness of the several materials as follows:

For country roads, suburban streets, and pleasure drives—broken stone. For streets having heavy and constant traffic—rectangular blocks of stone, laid on a concrete foundation, with the joints filled with bituminous or Portland cement grout. For streets devoted to retail trade, and where comparative noiselessness is essential—asphalt, wood, or brick.

Desirability. The desirability of a pavement is its possession of qualities which make it satisfactory to the people using and seeing it. Between two pavements alike in cost and durability, people will have preferences arising from the condition of their health, presonal prejudices, and various other intangible influences, causing them to select one rather than the other in their respective streets. Such selections are often made against the demonstrated economies of the case, and usually in ignorance of them. Whenever one kind of pavement is more economical and satisfactory to use than is any other, there should not be any difference of opinion about securing it, either as a new pavement or in the replacement of an old one.

The economic desirability of pavements is governed by the ease of movement over them, and is measured by the number of horses or pounds of tractive force required to move a given weight—usually one ton—over them. The resistance offered to traction by different pavements is shown in the following table:

Resistance to Traction on Different Pavements

KIND OF PAVEMENT	TRACTIVE RESISTANCE	
	Pounds per ton	*In terms of the load*
Asphalt (sheet)	30 to 70	$\frac{1}{67}$ to $\frac{1}{30}$
Brick	15 " 40	$\frac{1}{133}$ " $\frac{1}{50}$
Cobblestones	50 " 100	$\frac{1}{40}$ " $\frac{1}{20}$
Stone-block	30 " 80	$\frac{1}{67}$ " $\frac{1}{25}$
Wood-block rectangular	30 " 50	$\frac{1}{67}$ " $\frac{1}{40}$
Wood-block round	40 " 80	$\frac{1}{50}$ " $\frac{1}{25}$

Serviceability. The serviceability of a pavement is its quality of fitness for use. This quality is measured by the expense caused to the traffic using it—namely, the wear and tear of horses and vehicles, loss of time, etc. No statistics are available from which to deduce the actual cost of wear and tear.

The serviceability of any pavement depends in great measure upon the amount of foothold afforded by it to the horses—provided, however, that its surface be not so rough as to absorb too large a percentage of the tractive energy required to move a given load over it. Cobblestones afford excellent foothold, and for this reason are largely employed by horse-car companies for paving between the rails; but the resistance of their surface to motion requires the expenditure ot about 40 pounds of tractive energy to move a load of 1 ton. Asphalt affords the least foothold; but the tractive force required to overcome the resistance it offers to motion is only about 30 pounds per ton.

Comparative Safety. The comparison of pavements in respect of safety, is the average distance traveled before a horse falls. The materials affording the best foothold for horses are as follows, stated in the order of their merit:

(1) Earth, dry and compact.
(2) Gravel.
(3) Broken stone (macadam).
(4) Wood.
(5) Sandstone and brick.
(6) Asphalt.
(7) Granite blocks.

Durability. The durability of pavement is that quality which determines the length of time during which it is serviceable, and does not relate to the length of time it has been down. The only measure of durability of a pavement *is the amount of traffic tonnage it will bear before it becomes so worn that the cost of replacing it is less than the expense incurred by its use.*

As a pavement is a construction, it necessarily follows that there is a vast difference between the durability of the pavement and the durability of the materials of which it is made. Iron is eminently durable; but, as a paving material, it is a failure.

Durability and Dirt. The durability of a paving material will vary considerably with the condition of cleanliness observed. One inch of overlying dirt will most effectually protect the pavement from abrasion, and indefinitely prolong its life. But the dirt is expensive, it injures apparel and merchandise, and is the cause of sickness and discomfort. In the comparison of different pavements, no traffic should be credited to the dirty one.

Life of Pavements. The life or durability of different pavements under like conditions of traffic and maintenance, may be taken as follows:

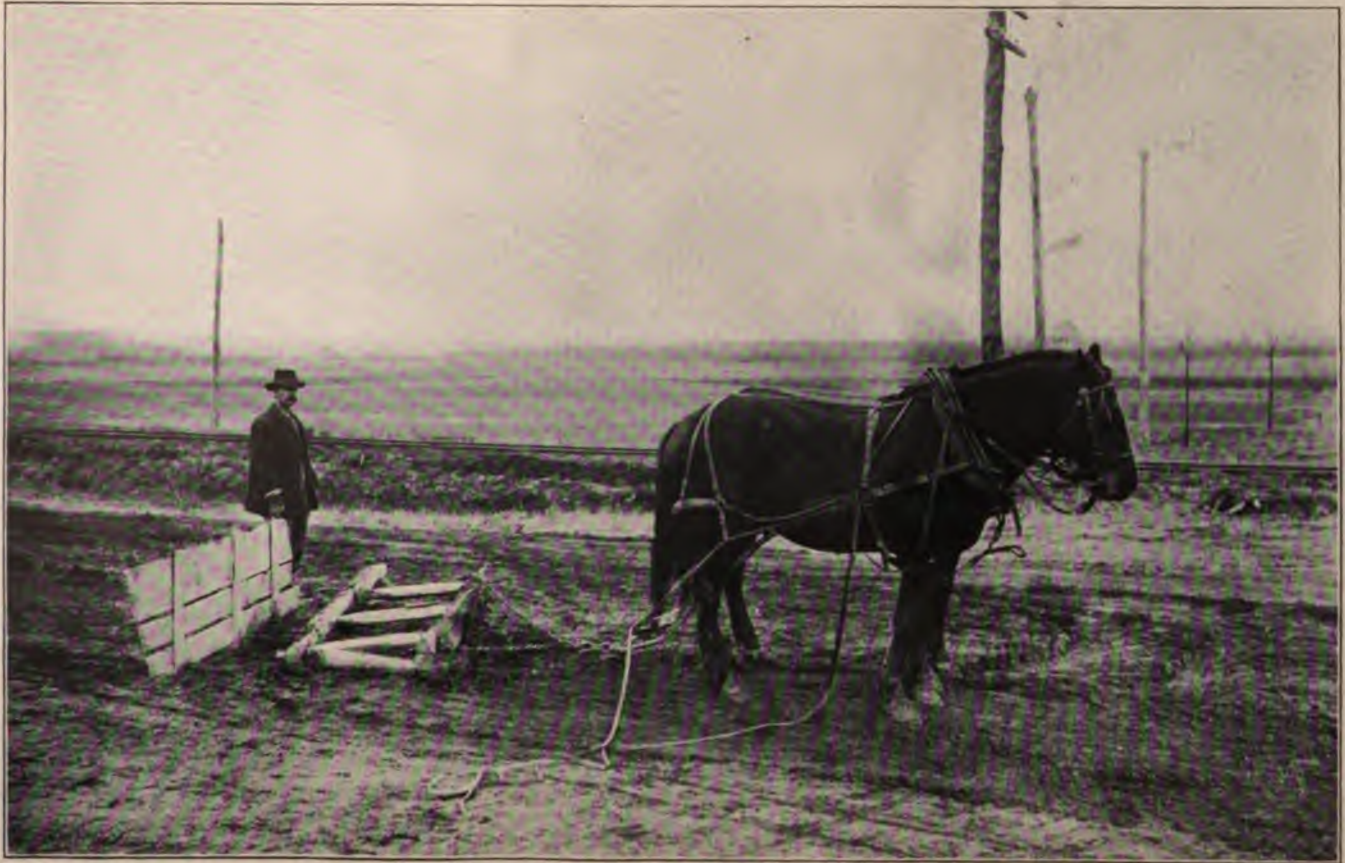
Life Terms of Various Pavements

MATERIAL	LIFE TERM
Granite block	12 to 30 years
Sandstone	6 " 12 "
Asphalt	10 " 14 "
Wood	3 " 7 "
Limestone	1 " 3 "
Brick	5 " ? "
Macadam	5 " ? "

Cost. The question of cost is the one which usually interests taxpayers, and is probably the greatest stumbling-block in the attainment of good roadways. The first cost is usually charged against the property abutting on the highway to be improved. The result is that the average property owner is always anxious for a pavement that costs little, because he must pay for it, not caring for the fact that cheap pavements soon wear out and become a source of endless annoyance and expense. Thus false ideas of economy always have stood, and undoubtedly to some extent always will stand, in the way of realizing that the best is the cheapest.

The pavement which has cost the most is not always the best; nor is that which cost the least the cheapest; *the one which is truly the cheapest is the one which makes the most profitable returns in proportion to the amount expended upon it.* No doubt there is a limit of cost to go beyond which would produce no practical benefit; but it will always be found more economical to spend enough to secure the best results, and this will always cost less in the long run. One dollar well spent is many times more effective than one-half the amount injudiciously expended in the hopeless effort to reach sufficiently good results. The cheaper work may look as well as the more expensive for the time, but may very soon have to be done over again.

Economical Benefit. The economic benefit of a good roadway is



SPLIT-LOG DRAG IN USE

Method of construction and operation is apparent from the picture. The advantages of the use of the drag are the maintenance of smooth, serviceable earth roads free from ruts and mudholes, at comparatively small cost of money or labor; reduction of mud in wet weather, and of dust in dry; and the banishment of weeds and grass.

comprised in its cheaper maintenance; the greater facility it offers for traveling, thus reducing the cost of transportation; the lower cost of repairs to vehicles, and less wear of horses, thus increasing their term of serviceability and enhancing the value of their present service; the saving of time; and the ease and comfort afforded to those using the roadway.

First Cost. The cost of construction is largely controlled by the locality of the place, its proximity to the particular material used, and the character of the foundation.

The Relative Economies of Pavements—whether of the same kind in different condition, or of different kinds in like good condition—are sufficiently determined by summing their cost under the following headings of account:

(1) Annual interest upon first cost.
(2) Annual expense for maintenance.
(3) Annual cost for cleaning and sprinkling.
(4) Annual cost for service and use.
(5) Annual cost for consequential damages.

Interest on First Cost. The first cost of a pavement, like any other permanent investment, is measurable for purposes of comparison by the amount of annual interest on the sum expended. Thus, assuming the worth of money to be 4%, a pavement costing $4 per square yard entails an annual interest loss or tax of $0.16 per square yard.

Cost of Maintenance. Under this head must be included all outlays for repairs and renewals which are made from the time when the pavement is new and at its best to a time subsequent, when, by any treatment, it is again put in equally good condition. The gross sum so derived, divided by the number of years which elapse between the two dates, gives an average annual cost for maintenance.

Maintenance means the keeping of the pavement in a condition practically as good as when first laid. The cost will vary considerably depending not only upon the material and the manner in which it is constructed, but upon the condition of cleanliness observed, and the quantity and quality of the traffic using the pavement.

The prevailing opinion that no pavement is a good one unless, when once laid, it will take care of itself, is erroneous; *there is no such pavement.* All pavements are being constantly worn by traffic and by the action of the atmosphere; and if any defects which appear are not quickly repaired, the pavements soon become unsatisfactory and are destroyed. To keep them in good repair, incessant attention is necessary, and is consistent with economy. Yet claims are made that particular pavements cost little or nothing for repairs, simply because repairs in these cases are not made, while any one can see the need of them.

Cost of Cleaning and Sprinkling. Any pavement, to be considered as properly cared for, must be kept dustless and clean. While circumstances legitimately determine in many cases that streets must be cleaned at daily, weekly, or semi-weekly intervals, the only admissible condition for the purpose of analysis of street expenses must be that of like requirements in both or all cases subjected to comparison.

The cleaning of pavements, as regards both efficiency and cost, depends (1) upon the character of the surface; (2) upon the nature of the materials of which the pavements are composed. Block pavements present the greatest difficulty; the joints can never be perfectly cleaned. The order of merit as regards facility of cleansing, is: (1) asphalt, (2) brick, (3) stone, (4) wood, (5) macadam.

Cost of Service and Use. The annual cost for service is made up by combining several items of cost incidental to the use of the pavement for traffic—for instance, the limitation of the speed of movement, as in cases where a bad pavement causes slow driving and consequent loss of time; or cases where the condition of a pavement limits the weight of the load which a horse can haul, and so compels the making of more trips or the employment of more horses and vehicles; or cases where conditions are such as to cause greater wear and tear of vehicles, of equipage, and of horses. If a vehicle is run 1,500 miles in a year, and its maintenance costs $30 a year, then the cost of its maintenance per mile traveled is two cents. If the value of a team's time is, say, $1 for the legitimate time taken in going one mile with a load, and in consequence of bad roads it takes double that time, then the cost to traffic from having to use that one mile of bad roadway is $1 for each load. The same reasoning applies to circumstances where the weight of the load has to be reduced so as to necessitate the making of more than one trip. Again, bad pavements lessen not only the life-service of horses, but also the value of their current service.

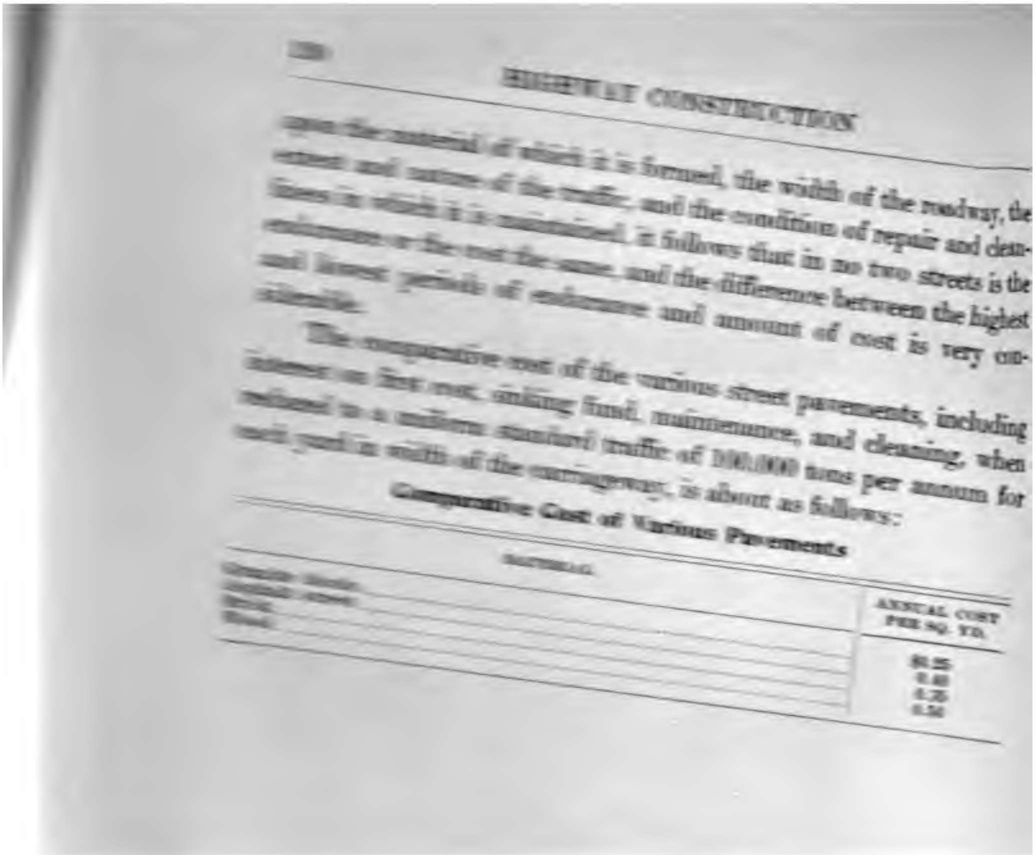
Cost for Consequential Damages. The determination of consequential damages arising from the use of defective or unsuitable pavements, involves the consideration of a wide array of diverse circumstances. Rough-surfaced pavements, when in their best condition, afford a lodgment for organic matter composed largely of the urine and excrement of the animals employed upon the roadway. In warm and damp weather, these matters undergo putrefactive fermentation, and become the most efficient agency for generating and disseminating noxious vapors and disease germs, now recognized as the cause of a large part of the ills afflicting mankind. Pavements formed of porous materials are objectionable on the same, if not even stronger, grounds.

Pavements productive of dust and mud are objectionable, and especially so on streets devoted to retail trade. If this particular disadvantage be appraised at so small a sum per lineal foot of frontage as $1.50 per month, or six cents per day, it exceeds the cost of the best quality of pavement free from these disadvantages.

Rough-surfaced pavements are noisy under traffic and insufferable to nervous invalids, and much nervous sickness is attributable to them. To all persons interested in nervous invalids, this damage from noisy pavements is rated as being far greater than would be the cost of substituting the best quality of noiseless pavement; but there are, under many circumstances, specific financial losses, measurable in dollars and cents, dependent upon the use of rough, noisy pavements. They reduce the rental value of buildings and offices situated upon streets so paved—offices devoted to pursuits wherein exhausting brain work is required. In such locations, quietness is almost indispensable, and no question about the cost of a noiseless pavement weighs against its possession. When an investigator has done the best he can to determine such a summary of costs of a pavement, he may divide the amount of annual tonnage of the street traffic by the amount of annual costs, and know what number of tons of traffic are borne for each cent of the average annual cost, which is the crucial test for any comparison, as follows:

(1) Annual interest upon first cost.......................... $
(2) Average annual expense for maintenance and renewal...
(3) Annual cost for custody (sprinkling and cleaning)......
(4) Annual cost for service and use........................
(5) Annual cost for consequential damages..................
Amount of average annual cost..........................
Annual tonnage of traffic.................................
Tons of traffic for each cent of cost......................

Gross Cost of Pavements. Since the cost of a pavement depends



upon the material of which it is formed, the width of the roadway, the extent and nature of the traffic, and the condition of repair and cleanliness in which it is maintained, it follows that in no two streets is the endurance or the cost the same, and the difference between the highest and lowest periods of endurance and amount of cost is very considerable.

The comparative cost of the various street pavements, including interest on first cost, sinking fund, maintenance, and cleaning, when reduced to a uniform standard traffic of 100,000 tons per annum for each yard in width of the carriageway, is about as follows:

Comparative Cost of Various Pavements

MATERIAL	ANNUAL COST PER SQ. YD.
[illegible]	$0.25
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]

REVIEW QUESTIONS.

PRACTICAL TEST QUESTIONS.

In the foregoing sections of this Cyclopedia numerous illustrative examples are worked out in detail in order to show the application of the various methods and principles. Accompanying these are examples for practice which will aid the reader in fixing the principles in mind.

In the following pages are given a large number of test questions and problems which afford a valuable means of testing the reader's knowledge of the subjects treated. They will be found excellent practice for those preparing for Civil Service Examinations. In some cases numerical answers are given as a further aid in this work.

REVIEW QUESTIONS

ON THE SUBJECT OF

BRIDGE ENGINEERING

PART I

---

1. Write a short history of early bridges.

2. Define: Truss, bridge truss, truss bridge, girders, and girder bridges.

3. Draw an outline of a through bridge, and also an outline of a deck bridge.

4. Make an outline diagram of a truss, and write the names of the various parts on the respective members.

5. Make an outline diagram of a Warren, Howe, Pratt, bowstring, and Baltimore truss.

6. Compute the weight of steel in a 130-foot highway bridge whose trusses are 16 feet center to center, given $W = 34 + 22b + 0.16bl + 0.7l$.

7. Compute the weight of steel in a deck plate-girder span of 100 feet. Loading, E 50. Given $W = 124.0 + 12.0l$.

8. What are *equivalent uniform loads?*

9. What is Cooper's E 40 loading?

10. Prove that the stress in a diagonal of a horizontal chord truss with a simple web system is $V \sec \phi$.

11. Prove that the chord stress is $M \div h$, where M is the moment at the point, and h is the height of the truss.

12. Prove that the load must be on the segment of the span to the right of the section to produce the maximum positive shear.

13. Compute the maximum positive and negative live-load shears in a 13-panel Howe truss, the live panel load being 40 000 pounds.

REVIEW QUESTIONS

ON THE SUBJECT OF

BRIDGE ENGINEERING

PART II

1. Write an essay of 200 words on the economic considerations governing the decision to build and the decision as to what kind of bridge to employ.

2. What determines the height and width of railroad truss bridges?

3. Draw a clearance diagram for a bridge on a straight track, and state what allowance should be made if the bridge is on a curve.

4. Describe a stress sheet, and tell what should be on it.

5. Make a sketch of a cross-section of a deck plate-girder, showing the cross-ties, guard-rails, and rails in place.

6. Make a sketch showing how tracks on curves are constructed.

7. What is the span under coping, the span center to center of bearings, and the span over all?

8. Design a tie for Cooper's E 50 loading.

9. If the end shear of a plate-girder is 394 500 pounds, design the web section, it being 108 inches deep.

10. If the dead-load moment is 8 489 000 pound-inches and the live-load moment is 30 610 000 pound-inches, design the flange, if the distance back to back of flange angle, is 7 feet $6\frac{1}{2}$ inches, it being assumed that the web does not take any bending moment.

11. If, in the girder of Question 10, above, the web were 90 by $\frac{7}{16}$-inch, design the flange, considering $\frac{1}{8}$ of the gross area of the web as effective flange area.

REVIEW QUESTIONS

ON THE SUBJECT OF

HIGHWAY CONSTRUCTION

PART I

1. Upon what does the ease with which a vehicle can be moved on a road depend?

2. What kind of a road surface offers the greatest resistance to traction?

3. How may the power required to draw a vehicle over a projecting stone be calculated?

4. What effect has gravity on the load a horse can pull?

5. Under what condition is the tractive power of a horse decreased?

6. What are the best methods for improving sand roads?

7 State briefly how earth is loosened and transported and the conditions under which each method is most advantageous?

8. What are the essential requisites for securing a good gravel road?

9. How should gravel roads be repaired?

10. State the considerations that control the maximum grade.

11. How should different grades be joined?

12. What considerations control the width of a road?

13. What is the essential quality of a stone used for road covering?

14. What should be the shape and size of broken stone?

15. For a light traffic road what thickness should the layer of broken stone have?

16. How should the foundation for the broken stone be prepared?

REVIEW QUESTIONS

ON THE SUBJECT OF

HIGHWAY CONSTRUCTION

PART II

---

1. How should the natural soil be prepared to receive a pavement?
2. In ramming blocks in the pavement, what point requires to be watched?
3. How is a sand cushion prepared for use?
4. What influences the durability of a granite?
5. How are rectangular stones laid on steep grades?
6. How is the surface and sub-surface drainage of streets provided for?
7. What are the principal objections to wood pavements?
8. What determines the best width for a street?
9. In filling the joints with gravel and bituminous cement, what should be the condition of the material?
10. What controls the maximum grade for a given street?
11. What varieties of wood give the most satisfactory results?
12. To what tests are stones intended for paving subjected?
13. Do cobblestones form a satisfactory pavement?
14. What properties should a stone possess to produce a satisfactory paving block?
15. How are expansion joints formed in a pavement?
16. What is the most suitable material for the foundation of a pavement?
17. Under what class of traffic may wood be used?
18. Upon what does the durability of a pavement depend?
19. What materials are employed for filling the joints between the paving blocks?

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